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Centre canadien de la technologie des minéraux et de l'énergie

ADVANCES IN CONCRETE TECHNOLOGY

Editor:

V.M. Malhotra

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ADVANCES IN CONCRETE TECHNOLOGY

Editor: V.M. Malhotra

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"Salhusbrua", a floating bridge
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PREFACE

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In 1980, Canada Centre for Mineral and Energy Technology (CANMET) published a book entitled "Progress in Concrete Technology". Nineteen internationally recognized experts at the invitation of CANMET contributed state-of-the-art papers on several aspects of aggregates, cement and concrete. Since then major developments have taken place in concrete technology. Examples include: the development of roller-compacted concrete, high-volume fly ash concrete, silica fume concrete, high-strength concrete for offshore structures, and new polymer composites. Also, new NDT methods have been developed, and accelerated test methods are being used increasingly for alkali-aggregate reactivity. Other new areas of concrete technology include advances in fibre-reinforced composites, shotcrete, fracture mechanics, and the activation of blast-furnace slag.

The purpose in publishing this volume, "Advances in Concrete Technology" has been to bring together in one volume in-depth reviews on these new developments. This was achieved by inviting 24 world renowned experts to write comprehensive state-of-the-art papers on their areas of expertise. To expedite timely publication, this volume has been printed directly from the manuscripts as received from the authors who are individually responsible for the views expressed. As a part of CANMET's concrete technology program, an international conference on Advances in Concrete Technology was sponsored jointly with American Concrete Institute and other organizations, and held in Athens in May 1992. This publication was distributed as a part of the conference proceedings.

It is hoped that this publication together with the one published in 1980 will contribute significantly to a better understanding of new technologies, and encourage their use by the concrete industry.

The Editor wishes to thank the authors for their efforts and co-operation. Particular thanks are extended to all those who assisted in the review of the manuscripts.

V.M. Malhotra
Editor
January 1992

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Concrete in the Year 2000

by

Adam Neville*

Synopsis: The development of special concretes for specific purposes and for repair will continue, using a wide range of ingredients and admixtures. Examples briefly considered are high performance concrete, roller-compacted concrete, high-volume fly ash concrete, and use of shotcrete, fibres and polymers. High-performance concrete is given prominence. Ordinary concrete, which represents the major part of concrete used in the world, sometimes suffers from lack of training of many of those involved, from failure to use the existing know-how, and from inadequate execution in some cases. In seeking to remedy the situation, the paper urges all concerned to do better by successfully applying the existing technology and available knowledge. The resulting improvement in the quality and, above all, in the durability of concrete will ensure that this excellent material, which has no economically viable competitor, will continue to be widely used into the next millenium.

Keywords: Aggregates; admixtures concrete; compressive strength; fibres and polymers curing; high-performance concrete; high-volume fly ash concrete; roller-compacted concrete; shotcrete.

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INTRODUCTION

As the year 2000 approaches, you might think that it is becoming increasingly easy to forecast the status of concrete in that year. In some ways, of course, this is true because not very much change can occur in the remaining few years of this millenium. In other ways, you cannot help thinking, and not just hoping, that a change must come about and must occur quickly if concrete is truly to serve its users: the population at large.

The nature of this change is the major subject matter of this paper. My aim is to show that the knowledge and capability to effect what I consider to be an essential change are already available. We are not awaiting the outcome of scientific investigations or the solution of complex technical problems. What is hampering progress is probably the lack of the incentive or the will to change.

In preparing a paper of this kind, it is appropriate to start with the preceding presentation on the same subject. This was a paper titled *Concrete for the year 2000* by Clyde E Kesler, published in 1980 in *Progress in Concrete Technology* edited by V Mohan Malhotra. My eminent and knowledgeable predecessor, as well as a good friend, opened his paper by observing that concrete "is confidently forecast to be indispensable in the future." This forecast continues to be valid, if only because no cheaper and equally versatile material has entered the market; nor is there any such material just round the corner. What a piece of luck for all of us involved in concrete!

If this is so, why have I, at the outset of this paper, referred to the need for change? Perhaps the best way of answering this question is to start by giving my view of the place of concrete in construction.

In fact, I see what one might call two concretes. They differ from one another in the purpose for which they are required. The first one is the ordinary day-to-day concrete for general use. By this I mean concrete for a wide range of structures, including roads and buildings, tunnels and bridges, dams and precast elements. You could say that this includes practically all the uses of concrete, so what is the second concrete?

As I see it, the second category of concrete is that which is required to have special properties; these may include a particular toughness or ductility, a particular permeability or resistance to chemical attack, or a particular strength well outside the usual range. I shall speculate on this second category of concrete later in my paper. At present, I propose to concentrate on what, for want of a better word, I shall call ordinary concrete.

ORDINARY CONCRETE

My definition of ordinary concrete is concrete which can be made using readily available materials and readily available know-how. These materials are not only portland cement, water and aggregate, but also a range of supplementary and cementitious materials and admixtures. No extensive further researcn is required to enable us to make ordinary concrete with satisfactory properties. But, in my experience, so much of the ordinary concrete does *not* actually have satisfactory properties. Admittedly, my view may have been soured by my experience as an expert witness and arbitrator. Clearly, it is only problems that land on the desk of a so-called expert, but there are many such desks and each one has a high pile of problems.

I hope therefore to be forgiven for the rather pessimistic tone of much of what follows. Of course, we are all aware of numerous concrete structures of various kinds and in all parts of the world which are excellent in execution and satisfactory and durable in performance. What I am trying to say is: would that *all* structures were like that.

If the right materials to make concrete are available, if the know-how is available, and yet there are problems, then what is the missing ingredient? My answer is: the quality of execution of the concrete construction. Before the reader calls out "not in my country", I hasten to say "of course, not." But somehow, these problems occur in many countries, and in some countries more often than others.

I now propose to give my views on what needs to be done to have a better ordinary concrete in the year 2000.

THE KNOW-HOW: THE CONCRETOR

First of all, we have the problem, who is it that works as the concretor? In most countries, this is not a recognized trade requiring specific qualifications. There is no apprenticeship to be served, no examination to be taken, and no status comparable with that of a mason, bricklayer, welder or steel fabricator. The consequence of this is that many of the concretors are less well educated, less well trained and less well motivated than other construction workers. This is not a new observation. In fact, remarks like this are made from time to time but they are not followed by action and by change.

I realize that there are all kinds of obstacles to introducing a trade of a properly trained and qualified concretor; if nothing else, it would cost money, and money is never available 'just right now'. But is it logical to expect a good quality product with uniform properties if its quality is in the hands of those who do not understand the technical consequences of what they are doing, or not doing, and if they have no incentive to gain such understanding?

The remedy is not difficult or complex but the change has be forced. A simple step would be lay down in, say, government contracts that a given proportion of the work force involved in concreting must have a certain qualification. With the passage of time, this proportion could be increased, and also there might be introduced the requirement of an additional qualification for some other, smaller, proportion of those involved in concreting. If adopted on government contracts, the system would readily extend to other construction.

The need for qualifications of concretors is one important requirement for better concrete in the year 2000. But, lest it be thought that I am 'getting at' the concretors, let me hasten to add another, complementary, requirement: the know-how of the engineer on the job.

THE KNOW-HOW: THE ENGINEER

Earlier on, I expressed the view that the know-how is available. It is, in the collected works of ACI and in the books of lesser mortals. But, much to my surprise and dismay, the know-how does not form part of a young engineer's education and training. I have ascertained that the best universities teach next-to-nothing about concrete technology. I sometimes suspect that not teaching something so ordinary, so simple as concrete (which could actually dirty your hands) actually adds to the status of the teaching institution.

Of course, the young engineer can learn on the job. But since the senior engineer is unlikely to be on the job to oversee concreting, the young one has to learn the hard way, by which I mean that he may well do so at the expense of unsatisfactory concrete the first time round. The remedy is not difficult, nor is it expensive. Proper understanding of what makes concrete 'tick' is required and this should be supplemented by training in concrete making and supervision. There is nothing wrong with dirtying your hands.

The know-how problem is not limited to young engineers. On reflection, perhaps the problem with the more senior people is not a lack of know-how but a certain unwillingness to accept newer (not new) methods and materials. One example is the use of superplasticizers. These are still considered by some - in certain countries, by many - as something fancy which is to be used only on special occasions. Many structures require concrete with a very low water/cement ratio, and the placing situation may make the use of a low-workability mixture difficult. The use of a superplasticizer seems an obvious solution. The use of a superplasticizer in conjunction with silica fume leads to very-high strength of the resulting concrete. In some other applications, such as shotcreting concrete containing fibres, a superplasticizer overcomes the problem of inadequate workability.

And yet, in some countries the use of superplasticizers is minimal. There seems to exist a reluctance to venture into what is perceived to be a somewhat exotic field. This attitude is not solely a consequence of the expense involved. After all, the *cri de coeur* of the cement manufacturers to the effect that cement is the best admixture still rings in many people's ears. I am all for enough cement but not too much.

Cementitious Materials

Of course, in the year 2000 and beyond, portland cement will continue to be the essential cementitious material, but it will be only one of the cementitious and latent-hydraulic ingredients. The notion that portland cement alone is better because it is "pure" will be discarded in favour of a blend of desirable materials.

At this point, it may be useful to remind ourselves that the products of hydration of cement act not only as a binder but also as a filler. After all, at low water/cement ratios, a significant proportion of the original cement remains unhydrated. Its presence contributes to strength because the remnants of the cement grains occupy space which otherwise would be voids; and it is the voids that reduce the strength of the concrete below the theoretical value for a fully solid material.

The problem is that clinker is a very expensive filler, and a partial solution is to use blended cements which contain a small proportion of an appropriate inert filler possessing a desirable fineness and suitable surface characteristics. Some sectors of the cement industry have in the past been opposed to what they considered to be an adulteration of pristine portland cement. Pure portland cement has served us well, but appropriate blends will serve us even better in the future. Fortunately, the more enlightened cement manufacturers see their future in terms of a flexible response to the demand for concrete with higher performance characteristics and to the availability of a range of materials which can be used as ingredients in a concrete mixture. I am convinced that the future lies in an increasing use of a wide range of cementitious and supplementary materials and their combinations.

The technical advantages of these materials in terms of thermal properties and permeability are well known; so is the control of alkali-aggregate reaction, with an improvement in our ability to assess the likelihood of the reaction and to proportion the mixture accordingly expected in the near future.

One argument in favour of the use of materials such as fly ash and blastfurnace slag is that, compared with portland cement, they represent energy saving. I am iconoclastic enough to believe that energy, like labour, can be expressed in terms of money. Thus, if energy is expensive, energy saving will follow by itself. But if, for example, in some location, blastfurnace slag is expensive, perhaps because it has to be imported from far afield, then the economic logic is against incorporation of slag in the mixture and a reduction in the content of portland cement; an engineer is normally duty bound to follow an economic argument.

The economic argument is also valid when considering the cost of construction of new cement plants. Because a high level of quality control is required, because the temperature involved is high, and because of a stringent need to avoid environmental pollution, the capital cost of a new plant is high. A typical figure is about $250 per tonne of annual capacity. This is high, considering that the end product - cement - should be a low-priced material. In many countries, especially in the developing world, the problem of a high capital cost is compounded by the need to import the plant without being able to offer an adequate return on the investment. It clearly follows that the use of any local supplementary cementitious or latent-hydraulic materials is advantageous. The range of these materials is increasing and includes now rice-husk ash.

Another compelling argument for the use of supplementary cementitious materials is the environmental consideration. This is a social duty and is also often reinforced by legislation. There is little doubt that converting potential heaps of blastfurnace slag or fly ash into ingredients of concrete is desirable. As these materials, appropriately used, are beneficial to concrete, I expect their use to continue to grow. For example, the range of usable fly ashes has already increased and so has the fly ash content as a proportion of the total content of cementitious and supplementary material. The pioneer work of CANMET in Canada has led to the development of mixtures containing a very high volume of fly ash and a high dosage of superplasticizer, with a consequent very low water/cement ratio. The resulting concrete has a very low heat of hydration (thus reducing the danger of thermal cracking), a very low permeability (and therefore offers a good protection to the reinforcement), and a satisfactory resistance to freezing and thawing. A much wider use of mixtures of this kind can be confidently expected. Indeed, I would be surprised, in the year 2000, to see much portland-cement-only concrete (even if I am still there in the year 2000 to look for it).

Materials and Structures

With respect to the education and training of engineers in the field of concrete, I find that a somewhat curious situation exists. For reasons which I have given earlier, many civil engineers, while knowledgeable about structures and structural design, lack a proper understanding of concrete. On the other hand, those who have a good grasp of 'concrete matters' are often not structural engineers but rather scientists. Since many problems in concrete construction involve a mismatch between the properties of concrete and the structure in which the concrete is used, it is highly desirable for the same person to be knowledgeable both about properties of the materials used and about structural behaviour.

This was certainly the situation in the past; the names of two such practitioners, one from each side of the Atlantic, readily spring to mind: Raymond E. Davis and Hubert Rusch. There are of course many others but their number steadily decreases.

Perhaps one reason for this decrease is that all the important discoveries about concrete and all the great work were done in the past so that concrete is no longer an exciting material. Be that as it may, the need for a sound and deep knowledge of concrete is as great as ever, and we should strive to re-create the breed of the concrete engineer.

THE 'IMAGE' OF CONCRETE

The patient reader - if indeed he or she has been patient enough to stay with me up to now - will accuse me of cataloguing concrete ills rather than foreseeing its future. The point I was trying to make is that there *are* many ills, malpractices or simply cases of crass ignorance, and our aim should be to remedy them by the year 2000. Sooner than that would be even better. Not to do so gives concrete a 'Cinderella' image.

To achieve what I consider to be essential for the production of quality concrete construction and for the removal of its bad name, it is absolutely necessary to effect the change which I am preaching. At present, concrete is viewed as necessary but not desirable. There is no glamour about it; nobody seeks you out at a cocktail party because you 'are in concrete'. It is within the power of those who 'are in concrete' to change the situation radically, and the change does not require expensive research or extensive investigations.

What is needed is a commitment to a consistent production of concrete of an appropriate quality for the use envisaged. One of the critical words is 'appropriate'; I shall refer later to using or transferring existing mixtures and practices to different situations where they are no longer appropriate. The second critical word is 'consistent'. Again, I shall refer later to the fact that, while the correct mixture may have been designed and while the correct mixture is batched some of the time, now and again things go wrong. In consequence, concrete of variable quality comes out of the mixer or the finished product is of variable quality. The economic consequences of this situation are enormous.

READY-MIXED CONCRETE

Over the years, there has been a steady and large increase in the proportion of concrete produced in ready-mixed plants. In densely populated areas, where most of the construction takes place within a small radius of a ready-mixed plant, and preferably more than one, there is everything to be said for buying concrete rather than making it yourself.

My reference to more than one plant conceals a barb. In some countries, there there have existed restrictive, and sometimes illegal, practices not only of price fixing but also of restricting the supply, other than at an exorbitant price, to certain mixtures. This situation makes it difficult for the purchaser to lay down specific requirements in terms of ingredients. Only competition, and not monopoly, will ensure an even greater use of ready-mixed concrete, and with it a better control of quality, which is so important for the future success of high-quality concrete and, indeed, of concrete in general. Application of anti-trust and similar laws will make these benefits possible.

SPECIAL CONCRETES

Let me now turn to the second category of concrete: concrete for special use. Because concrete is extremely versatile, its composition and shape being untramelled by the constraints of a factory or a process, it is possible to use tailor-made concrete for a wide range of applications. In each of these, there is some special requirement, and it has been increasingly possible to satisfy a wider range of applications.

The total volume of special concrete is unlikely to be large as compared with ordinary concrete; nevertheless, special concretes of various kinds are of great importance as they make it possible economically to achieve a number of types of structures, not all of them for special purposes.

Several papers in this publication will deal with the individual special concretes. Here it suffice to mention but a few examples.

Roller-Compacted Concrete

One of these special concretes is roller-compacted concrete. This is no-slump concrete with an extremely low content of portland cement and fly ash or blastfurnace slag; aggregate with a maximum aggregate of up to 150 mm can be used. The concrete is placed by earthfill methods, that is transported by truck and compacted by a heavy vibratory roller in layers which become bonded. This method of construction is much faster than the conventional methods. Because of this and also because of cost saving, roller-compacted concrete, the beginnings of which go back no more than ten years, is becoming an increasingly widely used material with great promise for future growth. Roller-compacted concrete has revitalized the construction of concrete gravity dams and associated works, and it can also be used in the construction of pavements. Greater use can be expected and thus a growth in the use of concrete.

A more general comment may be appropriate: the introduction of roller-compacted concrete is a splendid example of a transplant of technology from one field of technology, namely earth moving, to another, namely concrete. Such transfer is infrequent but is likely to become more common; a system parallel to geotextiles may well be developed before long.

Fibre-Reinforced Concrete

Fibre-reinforced concrete is another special concrete which is discussed at length elsewhere in this publication. For a large part, this material is used in repairs. More general structural use has not yet come about but, with a resolution of durability problems, we are bound to see an increased use of fibre-reinforced concrete in situations where its great freedom of form can be exploited.

Fibres can also be included in shotcrete, the benefit being a good cavitation and erosion resistance. Such shotcrete is used in linings and in repair work. Repairs represent an increasing activity in the field of concrete. In a way, it is sad because, as I argue in this paper, we have in our possession the means of making concrete which does not need repairing. Nevertheless, the situation being what it is, developments in shotcrete are of great benefit.

Use of Polymers

Other repair materials are also being developed. Some of these use a range of chemical admixtures; many others involve polymer composites and latex-modified concretes. Thermal and dimensional compatibility problems in the use of these materials are being resolved, and development of materials with a better bond to wet surfaces and with a greater ductility are expected. More widespread use in repair work is very likely.

Cost is of course an important consideration and so is the need for competent and skilled operatives, the latter need already discussed by me at some length. Automated batching and placing will make it easier to use polymer concrete.

As far as cost is concerned, considerable savings are possible through the use of monomers and resins obtained from recycled waste plastic; this is yet another example of combining environmental benefits with the use of a material from a new source. A cheaper material will widen the field of application, mainly in the manufacture of specialty building panels and other precast products. In situ applications are mainly in repairs and rehabilitation such as resurfacing and overlays. The improved resistance to chloride attack of reinforcement provided by latex-modified concrete is of course very valuable. Shotcreting polymer concrete with fibres is one example of development.

Many of these special concretes are discussed in individual papers in this publication, and I shall therefore not consider them further. I should point out, however, that a healthy development of special concretes continues and will continue. There is no doubt that by the year 2000 we shall have a wide range of specialized products and materials at our disposal. All this will be of great value to users of concrete but, of course, special concretes represent only a small fraction of all the concrete used.

High-Strength Concrete

There is one type of special concrete which deserves a more extensive reference: very-high strength concrete, perhaps more correctly called high-performance concrete. The second-mentioned apellation is more appropriate because this material has many highly desirable properties, including, above all, durability. Moreover, how high is very high? Once upon time, this was 40 MPa. Even as recently as 1980, Clyde Kesler, in his paper cited earlier, wrote: "Concretes with strengths in excess of 60 MPa have been used in a number of structures. A 75-MPa concrete has been experimentally used in two columns of one building." Today, 100 MPa can be consistently and regularly achieved, and 120 MPa is well within the capability of continuous production.

There are no secret or particularly expensive ingredients of this type of concrete. What is required to make high-performance concrete is the know-how of concrete technology and a meticulous care in execution. There are great advantages in the use of high-performance concrete, especially in high-rise buildings and long-span bridges as well as in offshore structures. It is therefore very likely that the use of such concrete will grow considerably: by the year 2000, the option of using this material will be regularly and beneficially exercised.

In a way, I am saying that high-performance concrete will merge into what I have called ordinary concrete. I am not suggesting that 100 MPa concrete should be routinely used; indeed, concrete with a much lower strength but adequately low permeability and adequately high resistance to freezing and thawing can be high-performance concrete. For every application, there is an appropriate strength and appropriate requirements of permeability, resistance to freezing and thawing, thermal properties, and so on, not forgetting of course the cost. What I mean is that the ability to make high-performance concrete is likely to be within the capability of most concrete making plants.

CONTINUING PROBLEMS IN USING CONCRETE

There are a few separate topics, each one individually small in scope but large in importance, which are brought forward at frequent intervals and on which no progress seems to have been achieved. In a paper of this kind, which, in considering the future of concrete, should not neglect negative aspects, it is appropriate to bring up these topics again.

At this stage, I cannot refrain from mentioning a paradox. Concrete is a structural material by virtue of the hydrated cement paste, which is cohesive, adhesive, and which possesses many desirable physical, chemical and mechanical properties. At the same time, nearly all the problems in concrete stem from the presence of hydrated cement paste: shrinkage, creep, thermal cracking, chemical attack, alkali-silica reaction.

Protection of the Reinforcement

In some, actually quite many, parts of the world there is a serious problem of corrosion of reinforcement in consequence of chlorides penetrating through the concrete and attacking the steel. Some years ago, we underestimated the aggressive nature of the environment and often simply transferred the concrete technology from the cooler part of the world without adapting it to the new conditions. Nowadays, we know that a successful protection of the steel requires an adequate cement content and a very low water/cement ratio. But, with such a mixture, there are workability problems, especially when the temperature is high, which it is in many parts of the world. Some engineers try to cope with the problem by a series of half-measures but shy away from accepting the need for a superplasticizer as axiomatic.

Some engineers are also ambivalent about the use protective barriers and integral waterproofing agents. The usual explanation, not to say excuse, is that none of these products has a long enough proven track record. Of course, this is true. The only way you can get long-term experience is by *doing* something. The 20-year behaviour of prestressed concrete was not known with any certainty until such concrete had been used for 20 years. You need experience but you also need judgement and courage.

An area about which I am not optimistic is the improvement in the corrosion resistance of the reinforcing steel itself. A particularly important problem is the attack by chlorides originating from sea water: so much of new construction takes place within a mile or so of the coast, especially in hot climates where most of the development occurs. Chloride attack is pernicious and, once the chloride ions have reached the surface of the steel, there is no simple and economic way of making the structure as good as new and ensuring that the problem will not recur. Instead of a cure, we have to go for prevention.

Replacement of steel by a non-corroding reinforcement, despite many attempts and some progress, does not seem to loom large on the horizon. If we have to live with steel, such as it is, then the remedy lies in improving the quality of the concrete protecting the steel. This is clearly possible if we change our view of what is good concrete. For the conditions being discussed, we shall have to think in terms of very low water/cement ratios: even 0.5 will have to be regarded with horror. There is no difficulty about using very low water/cement ratios with superplasticizers, and no difficulty about a very low permeability with appropriate mixture ingredients. But if we limit ourselves to using portland cement alone, then we cannot succeed.

The topic of reinforcement is a large one and is possibly outside the scope of the present paper. There is nevertheless one comment which I would like to make about the concept of covering the surface of the steel with a thin layer of protective material. This approach is likely to be fully effective only if 100 per cent of the surface of the steel is and remains so covered: there must be no pin holes, no cracks at bends, no baring in consequence of knocks, no scratches due to rubbing of one bar by another. In other words, the entire operation from the manufacture of a covered bar, through transporting and placing, right to the compaction of concrete requires very careful procedures. As these are not routine in concrete construction, and therefore not easy to achieve consistently, there is a risk of local corrosion here and there. Such corrosion could be of the pitting variety, fuelled by a large, protected cathode, and the consequences of such corrosion could be serious.

Better methods of protecting reinforcing steel may be developed in the future, but probably not by the year 2000. My concern is that we sometimes think that what is considered to be an effective protection of the steel surface removes the need for a concrete cover to the steel which is of highest quality in terms of preventing the access of aggressive salts to the surface of the steel. This is entirely erroneous.

Curing Concrete

Some of the products and methods referred to in the preceding section are new, and this may be why there is a reluctance to use them. But there is one procedure which is very old but is often honoured in breach: moist curing. In my opinion, inadequate curing is responsible for more problems with concrete than many another sin of omission.

There is no need to extol the virtues of curing in this paper. In theory, everyone believes in the importance of curing. In practice, hardly anyone finds it necessary to make or enforce the necessary effort, especially as its consequences are invisible and not measurable. To me, it is illogical that we are greatly concerned about the strength of concrete test specimens subjected to perfect curing, but who cares about the actual structure!

Above all, when concrete is exposed to an aggressive environment, the quality of the outer part is absolutely vital for the protection of the reinforcement, and it is the very same part that is most affected by the absence of curing.

Strength Tests

One of the continuing problems is the determination of the strength of concrete for the purpose of compliance. Everybody agrees that 28 days is too late to deal with non-compliant or suspect concrete, especially if concreting has continued during the intervening 27 days. Accelerated tests galore have been developed, and many of them are reliable in determining the strength of concrete under the particular conditions of testing. The trouble is that the major national specifications - and they are in fact used internationally - have not abandoned the 28-day strength test or even relegated it to a secondary or back-up role.

This situation cannot continue indefinitely. However, one obstacle is that the 28-day strength test is held to be simple and inexpensive to perform. The test may be simple but many of us have had experience of disputes arising from non-standard treatment of specimens, and doubts about capping or about the testing machine are not always unjustified. More than that, the standard strength test is expensive, especially if capping or grinding is required.

It is therefore unjustified to accuse any alternative test of being expensive or complicated. If the true cost, including the making, transporting and storing of test specimens, is taken into the account, and if the economic and technical consequences of a possible non-compliance are included, then it is worthwhile spending money on a rapid and consistent alternative test.

What do we want the test result to tell us? In my opinion, the strength of an arbitrary specimen, cured in an arbitrary manner, and tested at an arbitrarily selected age is not intrinsically valid. What is crucial to knowing whether the concrete complies with the requirements previously established to represent an appropriate mixture for the job in hand is the water/cement ratio and, in many cases, the cement content. What must come, and what, I am confident, can be done, is to develop a truly rapid test for these parameters. To achieve this objective, Standard writers of the world unite!

Testing in general is an area where considerable developments are certain to occur, if only because of the commercial interest of equipment manufacturers. Non-destructive test methods are particularly desirable. Among those which exist technically but need refinement are the impact-echo and pulse-echo techniques as well as the use of radar.

Moisture Content of Aggregate

A problem, thought by many to be trivial, is the control of the moisture content of aggregate. Not infrequently, I have come across situations where the variability of concrete, as produced, in terms of workability and strength, is due to an inadequate allowance for the variation in the moisture content of the aggregate. Sure enough, the situation is quickly remedied by adjustments to the water put into the mixer, but not quickly enough to avoid several batches of unsatisfactory and non-compliant concrete.

Not much new development is required to ensure a continuous control of the moisture content of aggregate. A plant which does not ensure such control should not be allowed to be used for the production of concrete other than that destined for the back yard.

It is worth noting that a strict control of all the free water put into the mixer would be of direct value in the application of the method of ensuring compliance discussed in the previous section. If the total free water content of every batch of concrete is known reliably, then the water/cement ratio of all the concrete is likely to be properly constant, and thus compliance with the specified value will probably be ensured.

AGGREGATE

Much of what I have said earlier is based on the premise that concrete is as good as the binder, but it is also true that good concrete requires appropriate aggregate. Fortunately, the range of aggregates which can be used is wide, and it is the task of the person proportioning the mixture to select and blend appropriate aggregate fractions. Indeed, in the future the supply of readily available top-quality aggregate will diminish because environmental constraints rightly make it undesirable to open new pits or to denude banks and shores.

What we shall have to do is to manufacture more aggregate, using industrial by-products and waste materials of various kind. A plentiful source is the residue from garbage incinerators and processed sludge. Disposal of these is a problem in itself, so that there is a commonality of interests in converting the undesirable into a marketable product. Much of the technology for doing so is already available but the necessary incentive, namely economic advantage, is generally absent. While I am, as a rule, not in favour of compulsion by legislation, this is surely a special case. What makes it special is the environmental aspects of the problem both in terms of waste disposal and aggregate production. I expect that tax legislation will lead to a much greater use of waste as a source of aggregate.

Another desirable source of aggregate is recycled concrete. Here again, the problem of disposing of concrete rubble can be neatly solved by processing it into a form suitable for use as aggregate. This is already done in some places and the usage of recycled concrete is very likely to become more widespread.

IS THERE A FUTURE FOR CONCRETE?

The future use of concrete is likely to undergo some changes. In those parts of the world which still lack an adequate infrastructure of transportation, energy generation, industrial buildings and housing, large quantities of concrete will continue to be used. On a world scale, this will represent a very large volume of concrete.

On the other hand, in that part of the world which has for many years been a large concrete user, there will be a slowing down. It is not only that we shall not continue to construct large dams and numerous bridges, but even the need for high rise offices will decrease as people work more from home and possibly more often live away from large conurbations.

Of course, concrete will continue to be used but, I think, less of it will be seen. The dreadful sobriquet of 'concrete jungle' has not gone away. There are some outstanding and beautiful concrete structures but they are greatly outnumbered by massive, drab, unattractive, if not outright ugly, concrete blocks, often grouped into depressing collections where uniform dark grey is broken up only by varying shades of weathering and staining.

These are strong words, and nothing would give me greater pleasure than to see them disproved by the year 2000. But, in the meantime, I expect less concrete to meet the eye, even though, as a building and structural material, concrete will still be there.

I am aware of the fact that, on the whole, my paper is rather critical and sombre, if not pessimistic. I have to admit that my views are coloured by my experience of investigation of failures, of expert witness work in disputes about non-compliance and non-performance, and of arbitration. The amount of money involved in some of these disputes is staggering, even though I have not yet seen a single claim which exceeds one billion dollars. The variety and range of structures is also enormous: buildings, highways, tunnels, airports, bridges, underground construction, storage tanks, spillways, gas processing plants, refineries, harbour works, marine structures. Does there exist any other type of concrete structure?

In some cases, the extent of damage is very large. Inevitably, the cost of repair is sky-high, and so is the cost of determining the responsibility for the defects. First class legal and engineering brains are engaged and the cost is commensurate with their quality.

The cost of the legal action, as well as of the repairs, ultimately falls upon all of us. If it is the government that foots the bill, the ultimate payer is the taxpayer. If a contractor becomes bankrupt, his creditors suffer, but also the perceived risk raises the cost of future construction. Thus, a badly executed structure has to be paid for twice.

What is sad to say is that the original cause - there are often several - of the problems is often minor in itself, although large in consequences. It may be no more than lack of a proper control of moisture content of the aggregate; it may be an inadequate mixing time; it may be the use of a particular type of mine tailings as aggregate; or a mistaken dosage of an admixture; or simply poor or careless workmanship. After all, somebody thinks, "it is only concrete - any fool can do it". The trouble is that he does.

Where do we go from here to ensure that the year 2000 heralds the dawn of high quality, reliable concrete throughout the world?

First, it is obvious that the concrete has to be well executed by competent concretors under competent supervision.

Second, we should expose the problem to the public so that it appreciates the importance of concrete and appreciates the economic value of good quality concrete, which of course has to be paid for.

Third, we should carefully select the type of concrete according to need, a significant factor being the conditions of exposure. A proper selection can be made only by a knowledgeable engineer.

Fourth, let us 'jazz up' the image of concrete and of the people who make it - from the concretor to the designer. Some of this will follow from the second and third points above. But we should all work at it and not hide our light under the bushel. With the wealth of talent of people who 'are in concrete' and with dedication, a good concrete world in the year 2000 is achievable.

APOLOGIA

This paper can be perceived as pessimistic but it would not be correct to surmise that I do not believe in the future of concrete. There is no doubt that concrete is a most useful and versatile material, and it is highly unlikely that it will be supplanted by any other material in the foreseeable future. My pessimism stems from the fact that we have not improved the quality of ordinary concrete as much as we could by simply using existing knowledge and technology. What I have tried to do is to ram home the message that we have in our possession all the means necessary to do better in the use of ordinary concrete. So there is a method in my madness: I am trying to incite those concerned with concrete construction into doing better, and then better still.

HIGH-STRENGTH CONCRETE

By Odd E. Gjorv

Synopsis: Based on current technology it is now possible to produce high-strength concrete with compressive strengths far beyond what can be utilized by the current structural design practice. Thus, for high-quality natural mineral aggregates compressive strengths of up to 230 MPa can be produced. If the mineral aggregate is replaced by high-quality ceramic aggregate, compressive strengths of up to 460 MPa can be achieved. Even with lightweight aggregate, compressive strengths of more than 100 MPa, with a density of less than 1900 kg/m^3, can be obtained. Very often it is not the improved strength which is the primary objective but rather the overall improved performance. In the present paper, a brief summary of the most recent developments on high-strength concrete based on mineral aggregates is presented.

Keywords: Abrasion resistance; cement; chemical admixtures; chemical resistance; concrete mineral admixtures; corrosion; curing; deformation; durability; fire resistance; high-strength concrete; mechanical properties; microstructure; mixture proportioning; resistance; superplasticizers; workability.

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INTRODUCTION

In recent years more rapid developments in the field of concrete technology have taken place. Increasing construction challenges in combination with new inovations in materials and construction techniques have strengthened the position of concrete as a construction material. Having been an empirical technology, the concrete technology is now entering into a high-technology profession, with a potential for utilization far beyond the traditional construction industry. An increasing interest for better scientific knowledge in concrete technology has correspondingly taken place.

Although most concrete codes still have an upper grade limit of 65 MPa for structural utilization of concrete, new codes have recently been introduced with an upper limit of up to 105 MPa(1). Based on current technology, however, it is now possible to produce high-strength concrete with compressive strengths far beyond what can be utilized by the current structural design practice. Thus, for high-quality natural mineral aggregates, compressive strengths of up to 230 MPa can now be produced(2). If the mineral aggregate is replaced by high-quality ceramic aggregate, compressive strengths of up to 460 MPa can be achieved(3). Even with lightweight aggregate, compressive strengths of more than 100 MPa, with a density of less than 1900 kg/m^3, can be obtained(4).

Along with the rapid development of concrete technology, the definition of high-strength concrete has also been changing. Thus, in the 1950's concrete with a compressive strength of 35 MPa was considered high strength. In the 1960's concrete with 40 to 50 MPa compressive strengths was used commercially. In the early 1970's, 60 MPa concrete was being produced. Since the compressive strength depends on both type of specimen and density of concrete, concretes with a compressive strength of more than 50-60 MPa are now normally considered high strength.

There has also been a rapid development in published literature on high-strength concrete. Based on 172 references, American Concrete Institute (ACI) published a State-of-the-art-report in 1984(5). In 1985 both ACI(6) and Materials Research Society(7) published proceedings of two international conferences on high-strength concrete. In 1987 the proceedings from the First International Symposium on Utilization of High-Strength Concrete was published(8), followed by the proceedings of the Second International Symposium in 1990(9). In 1988 the British Cement Association published a literature review of high-strength concrete properties(10). In 1990 a new State-of-the-art-report on high-strength concrete was published by FIP-CEB(11). At the time being RILEM is also preparing a comprehensive State-of-the-art-report on the materials aspects of high-strength concrete(12). In the following, a brief summary of the most recent developments on high-strength concrete based on mineral aggregates is presented.

SELECTION OF MATERIALS

Aggregates

Since about three quarters of the concrete volume is occupied by aggregate, it is not surprising that the aggregate quality generally is of considerable importance to the concrete properties. For high-strength concrete, however, experience has shown that the quality of the aggregate is one of the main limiting factors both for obtaining good workability and high strength.

The key for mix design of high-strength concrete is a low w/c ratio. To obtain this with a relatively high cement content and water-reducing admixtures, the water requirement of the aggregate component for a given consistency of the concrete mixture has to be as low as possible. This water requirement is primarily goverened by particle size distribution, particle shape and mineral composition of the fine aggregate. By a combination of new cementitious materials and superplasticizers it is possible to produce so strong a cement paste that the aggregate and particularly the coarse aggregate, becomes the weakest link in the chain under axial compression.

Although aggregate mineralogy and petrography predominantly determine the intrinsic strength of the aggregate(13,14), other factors such as blasting and crushing during processing also play an important role. During blasting and crushing operations, damage to the microstructure (e.g. microcracking) of the aggregate may occur. This is probably the main reason why a smaller maximum size aggregate yields a stronger concrete for a given aggregate source. A maximum size of 10 to 14 mm is often preferred.

In the concrete composite, the aggregate-cement paste bond also plays an important role to the interaction between the two components. Thus, in addition to the mineralogy, the aggregate shape and smoothness also are important factors. For cement pastes with very low w/c ratios, the interfacial bond or the transition zone between the aggregate and the cement paste, is normally so strong that the elastic properties of the aggregate become more important for the performance of concrete.

Aitcin and Mehta(14) have demonstrated the importance of the elastic properties of the aggregate by studying the shape of the hysteresis-loop during testing of the elastic modulus of concrete. Concrete specimens showing thick hysteresis loops and significant unrecoverable strain on unloading are indicative of weakness in the transition zone or the aggregate particles. A narrow hysteresis loop and zero residual strain are indications of an aggregate type that not only is strong itself, i.e. free from internal defects, but is also capable of forming a strong transition zone. For instance, fine-grained rock with rough surfaces are likely to form strong physical-chemical bonds with the hydrated cement paste.

The above testing procedure has provided a new basis for evaluating the potential of an aggregate for application to high-strength concrete.

Cement and mineral admixtures

Both the chemical and mineral composition as well as the particle size distribution of the cement are important factors to be considered in the selection of a cement for high-strength concrete.

In addition to the strength development, the type of cement is also important for the water requirement and workability of the mix, for which it is primarily the C_3A content and the particle size distribution which are the main controlling factors(15). The compatability with chemical admixtures is also important for the fresh concrete workability.

The mineral composition and the fineness of the cement are factors that influence both early and final strength. Although C_3S contributes both to a high early and high final strength, a high C_2S/C_3S ratio is normally considered to be beneficial for a high final strength(16). Thus, the special high-strength Norwegian cement which has been developed for offshore concrete structures in the North Sea, has a C_2S/C_3S-ratio of 0,54 compared to 0,38 which is typical for a Norwegian ordinary portland cement.

A reduced heat of hydration is also considered to be beneficial for high ultimate strength(16). Thus, if high-early strength is not needed, a cement with characteristics which contribute to low heat of hydration, such as less C_3A and C_3S in addition to low fineness and alkalies, will have the potential for high ultimate strength.

When a portland cement hydrates, large quantities of calcium hydroxide form, which do not contribute to the strength development. On the contrary, the large crystals of calcium hydroxide only represents weaknesses both at the aggregate-cement paste interface as well as in the bulk phase of the cement paste. By introducing mineral admixtures such as fly ash or silica fume, the calcium hydroxide is transformed during the pozzolanic reaction to the strength-contributing constituents, namely calcium silicate hydrates.

Although fly ash also contributes to a reduced water requirement, silica fume is a much more efficient pozzolanic material(17). In addition, silica fume represents a valuable range of very fine particles which fill out the voids inbetween the larger cement particles. Thus, for most binder combinations, the use of silica fume is reported to provide the only way of producing concrete with a strength level exceeding 80 MPa(11).

Chemical admixtures

In order to sufficiently reduce the water content of the mix and still maintain an acceptable workability, high-range water-reducing agents (superplasticizers) have to be used. In addition, set-retarding agents and air-entraining admixtures may also be needed.

Superplasticizers are organic products or combinations of organic and non-organic compounds. They are surfactants, or surface-active admixtures, consisting mostly of sulfonated salts of melamine or naphtalene formaldehyde condensates. Some formulations may contain set-retarding, set-accelerating or de-foaming chemicals. An accelerated superplasticizer may be used to compensate for retarding actions of active ingredients present in some superplasticizers such as modified lignosulfonates. A modified retarding superplasticizer can be useful in low-slump concrete where the fluidity loss is sharp.

A reduction in fluidity which is often observed as a physical-chemical interaction between a superplasticizer and the cement, is usually accosiated with the formation of ettringite. Because of its low solubility, crystals of ettringite begin to precipitate into the solution. Free water in the system gets entrapped among precipitating crystals, and also gets adsorbed onto the ettringite crystals. The resulting reduction in free water causes a loss of workability. Therefore, portland cement with a high C_3A content can exhibit very rapid slump loss in the presence of a superplasticizer containing large amounts of free sulfate ions. The loss of slump is also greater at elevated temperatures(18). If a set-retarding admixture or a modified set-retarding superplasticizer is not used, and lasting high fluidity is desired, the superplasticizer or a portion of it may have to be added and remixed in the automixer at the job site.

Superplasticizers based on sulfonated naphthalene polymer and modified lignosulfonate may entrain some air in the concrete, wheras sulfonated melamine polymers usually do not cause any significant increase in air content. Also, some superplasticizers can cause difficulties in securing a proper air void system needed to produce good frost resistance. Siebel(19) observed that in the presence of a superplasticizer, the entrained air bubbles coalesced and formed larger voids in such a way that the spacing factor increased. As a result, the dosage of an air-entraining admixture was significantly increased in superplasticized concrete to secure adequate frost resistance.

CONCRETE PRODUCTION

Mixture proportioning

The basic relationship between reduced w/c ratio or porosity and increased compressive strength is also valid for high-strength concrete. However, for normal-strength concrete an improved strength very often is a question of reduced w/c-ratio in the form of increased cement content, wheras for high-strength concrete it is primarily a question of balancing a reduced water content with an adequate workability. Therefore, selection of concrete components and mix design is more critical for high-strength concrete, and more attention must be given also to the concrete mixing efficiency and mixing procedure.

In order to produce a high-strength concrete the w/c ratio is normally in the range of 0,20 to 0,40. The importance of type of cement and type of aggregate for a reduced water requirement has already been discussed. However, in order to obtain acceptable workability for the mix with very low w/c ratios, the overall particle size distribution in the finest range becomes more crucial. In particular, the packing of ultra-fine particles inbetween the cement particles becomes more important(20). In order to avoid agglomoration of the finest particles, a proper dispersion by use of superplasticizers also becomes more important (Fig. 1).

Extrepolation of experimental results indicate that a properly composed cement paste of maximum density will give an ultimate compressive strength of up to approximately 500 MPa (Fig. 2). Therefore, in a concrete composite it will ultimately be the strength of the aggregate and the aggregate-cement paste bond which are the strength limiting factors.

Even for normal-strength concrete the mix design is not an easy process to control because it normally involves a balancing of various conflicting requirements. Extensive laboratory testing must often be carried out before a satisfactory mix design is obtained. For high-strength concrete the proportioning of the component materials becomes even more difficult primarily due to the conflicting requirement of a very low water content and adequate workability.

In the literature various procedures for mix design of high-strength concrete have been proposed. In 1974 Blick et al.(21) published some general guidelines for selecting a maximum aggregate size. They also pointed out that for a given aggregate size, the fine-coarse aggregate ratio should be reduced due to the high cement content. More recently, Mehta and Aitcin(22) proposed a step-by-step procedure based on a fixed paste content of 35% and tabulated mixture proportions depending on the desired strength. In this procedure the workability is only controlled by the use of superplasticizers, which in practice may be difficult. Addis and Alexander(23) have proposed another method based on the ACI approach for concrete proportioning. Also in this method, more emphasize is given to aggregate aspects than to the cement paste, the composition of which remains more unclear.

Based on the Farris(24) mathematical model for calculating the viscosity of suspensions, de Larrard(25,26) has proposed a method for mix design with respect to both strength and workability and at the same time keeping the paste volume as low as possible.

During mixing of the concrete, attention must be given to the sequence in which the component materials are added to the mixer and also to the efficiency of mixing. In particular, the rate of admixture discharge and time of addition can alter the effectiveness of the admixture, as well as the water demand in the mix(27).

As part of the quality assurance and quality control of high-strength concrete production, it is extremely important to maintain a proper control of the constituent materials. In particular, attention should be given to the grading and moisture content of the fine aggregate. As can be seen from Fig. 3, an uncontrolled variation of 1% moisture content in the fine aggregate will have a much bigger impact on the strength at a very low w/c ratio compared to that at a more normal or high w/c ratio.

Workability and placing

For high-strength concrete, the very low w/c ratios, silica fume and superplasticizers affect the fresh concrete properties in such a way that neither the "static" nor the "dynamic" behavior can be compared to that of normal-strength concrete. Therefore, the conventional test methods such as slump or flow table do not predict the respons to compaction in the usual way. The conventional methods are still valid for giving an indication of whether the workability is plastic, heavy plastic or fluid, but they do not provide any information about the flow properties of the concrete. Thus, concrete with varying contents of silica fume and superplasticiers may have the same slump but very different flow properties. Therefore, a more basic approach is needed for testing and describing the workability of high-strength concrete.

In testing the fresh concrete workability by use of a viscometer (Fig.4), the concrete normally behaves like a Bingham liquid(30) as shown in Fig. 5, where:

$$\text{Torque} = \tau_0 + \mu \cdot \text{Rotation speed} \qquad (1)$$

By measuring the torque on the stationary inner cylinder while the outer cylinder is rotating at various speeds, the values of τ_0 and μ can be determined. These parameters define the workability of the fresh concrete, where:

τ_0 is a measure of the force necessary to start a movement of the concrete ("yield strength" or "flow resistance").

μ is a measure of the resistance of the concrete against an increased speed of movement ("plastic viscosity").

By using these two parameters it is possible to provide a more general and basic characterization of the fresh concrete behavior. If the reference point in a τ_0 - μ diagram is moved in various directions as shown in Fig. 6, it indicates that the mix either becomes more wet, stiff or viscous. By adding admixtures such as plasticizers (P), superplasticizers (SP) or air entraining agents (L) compared to that of water, the reference point moves in various directions as shown in Fig. 7.

Very little basic information on the rheological properties of high-strength concrete exists. However, by adding increasing amounts of a superplasticizer to the concrete, a dramatic increase in viscosity can be observed (Fig.8). In the same figure, the effect of time on the stiffening can also be seen. By successively replacing the cement with increasing amounts of silica fume (Fig.9), in the first stage the viscosity is strongly reduced, while the yield strength is not affected. Above a certain threshould value of silica fume, however, a substantial increase of yield strength takes place, while the viscosity also starts to increase again. The threshold value depends on the cement content.

Since the high-strength concrete normally is characterized by a high viscosity, it is often felt on the job site that the mix is unstable and "sticky", in such a way that different compaction and casting techniques may be required. As can be seen from Fig. 7 the addition of air entrainment has a reducing effect on the viscosity. Such a beneficial effect of air entrainment has been utilized in the slipforming of silos made with high-strength concrete[33]. By utilizing this effect of air entrainment, however, attention must be paid also to the strength loss.

The use of plasticizers will also affect the period of set and the rate of workability loss. As already discussed, set-retarding admixtures or a redosage of admixtures on the construction site is often necessary to overcome this problem.

Curing

Information on heat of hydration and strength development is important both from a construction point of view as well as from the point of view of ensuring the quality of the final product. High curing temperatures may lower the final quality and icrease the risk of thermal cracking.

For concretes with low w/c ratios, however, Fig. 10 demonstrates that the heat of hydration is substantially reduced due to lack of water. As can be seen from the figure, the overall heat contribution from the pozzolanic material is also very low. Thus, the presence of a pozzolanic material in high-strength concrete is beneficial also from the standpoint of reducing the heat of hydration.

Experience so far indicates that high-strength concrete with silica fume withstands high curing temperatures better than that of ordinary portland cement in normal-strength concrete. Although experiments have shown that the compressive strength of high-strength concrete may not be reduced even for curing temperatures of up to 85°C, an increased permeability has been observed(35).

When the w/c ratio becomes very low, the relative speed of strength and heat development is increased, which may make a normal setting cement or silica-blended cement to behave as a rapid setting cement (Fig.11). Attention must be given, therefore, to the development of critical temperature gradients.

Generally, high-strength concrete is far more exposed to the risk of plastic shrinkage during the period of setting compared to that of normal-strength concrete. This is a consequence of the reduced water bleeding due to the low water content in combination with high amounts of cement and pozzolanic materials such as silica fume.

For all concretes, a proper curing procedure is essential for ensuring the quality potential. Thus, water curing provides better conditions than sealing with plastic sheets or curing compounds. Bentur and Goldman(36) observed that the effect of poor curing on the skin properties was equally detrimental with portland cement concretes of 62 and 78 MPa as with a 108 MPa concrete containing silica-fume.

MICROSTRUCTURE

One of the main characteristics of high-strength concrete compared to that of normal-strength concrete is the more uniform and homogeneous microstructure. When the portland cement is combined with the ultrafine particles of silica fume in low w/c ratios, the microstructure of such systems consists mainly of poorly crystalline hydrates forming a more dense matrix of low porosity. With increasing silica fume content, increasing content of the calcium hydroxide is transformed into calcium silicate hydrates (Fig. 12), while the remaining calcium hydroxide tends to form smaller crystals compared to that in pure portland cement pastes. From Table 1 it can also be seen that the calcium/silicate ratio of the hydrates decreases, which allows the hydrates to incorporate ions, such as alkalis and aluminum. As a consequence, it is proposed that an increased resistance to aggressive ions and alkali-aggregate reaction is obtained. Also, the electrical resistivity is increased.

One of the main effects of silica fume in high-strength concrete is to improve the microstructure of the transition zone between the aggregate and the cement paste. For normal-strength concrete on a pure portland cement basis, the transition zone around the aggregate, which is 20 to 100μm wide, has a very different microstructure compared to that of the bulk matrix(39,40). This transition zone which is inferior in quality and thus leading to a more poor bond between the aggregate and the cement paste, is typically characterized by the following key elements:

(1) The transition zone is richer in calcium hydroxide and ettringite than the bulk phase, and the calcium hydroxide is oriented. A rim of massive calcium hydroxide can often be observed around the aggregates.

(2) The porosity of the transition zone is greater than that of the bulk phase, and a gradient in porosity can be observed with a declining trend as the distance from the aggregate surface increases.

For normal-strength concrete, the special microstructure of the transition zone is apparently related to the formation of water-filled spaces around the aggregates in the fresh concrete, due to internal bleeding and to a "wall effect", which interferes with inefficient packing of the cement particles around the aggregates. Calcium hydroxide and ettringite are known to preferably grow in large pores, which accounts for the greater contents of these phases in the transition zone(41,42). Also, this zone is higher in w/c ratio relative to the bulk(43) and is therefore characterized by higher porosity.

When silica fume is introduced into the system, and in particular in high-strength concrete, considerable changes in the microstructure of the transition zone take place. Regourd et al.(44,45) and Aitcin et al. (46,47) observed that high-strength concrete with silica fume was not as crystallized and porous as that of normal-strength concrete and all of the space in the vicinity of the aggregate was occupied with amorphous and dense calcium silicate hydrates. Also, direct contact was formed between the aggregate and the calcium silicate hydrates rather than with calcium hydroxide as in normal-strength concrete. Scrivener et al.(48) quantified the interfacial microstructure and demonstrated that in high-strength concrete with silica fume, the porosity of the transition zone was practically eliminated (Fig. 13), and practically no gradient in porosity was observed, in contrast to normal-strength concrete.

The above qualitative and quantitative observations indicate that in high-strength concrete the typical dense microstructure of the bulk matrix extends up to the aggregate surface, in such a way that the inhomogenity of the transition zone is largely eliminated. It is now well documented that this improved microstructure is closely related to the improved performence of high-strength concrete.

As demonstrated in Fig. 14 the strength of high-strength concrete with silica fume is greater than that of the matrix, indicating a contribution of the aggregate to the strength in contrast to normal-strength concrete, whereas the concrete strength at 0% silica fume content is slightly weaker than that of the paste strength. The improved bond also affects the behavior of high-strength concrete during loading. Thus, the stress-strain curve is linear up to about 80% of its strength (47,50-52) in contrast to about 40% in normal-strength concrete, and the extent of the hysteresis on unloading is becoming small. Also, the steel-concrete bond is improved by the same mechanism(47), and the improved microstructure of the steel- cement paste transition zone improves the pull-out strength of embedded steel bars(53).

MECHANICAL PROPERTIES

Compressive strength

For normal-strength concrete a conversion factor between cube and cylinder strength of 0,8 is often applied, while for high-strength concrete this factor seems to change to a constant difference varying from 10 MPa in the Norwegian Concrete Code(1) to 15 MPa in the Model-Code MC-90(55). However, no universial conversion factor for different test specimens exists.

Held(56) has investigated the conversion factor for a number of mixes with the cylinder and cube strength varying from 50 to 95 MPa and 60 to 110 MPa, respectively (Fig. 15 and Table 2). Smeplass(57) has also investigated the conversion factor for high-strength concrete (Table 3). When mixture proportioning parameters such as paste-aggregate ratio, silica-fume content, fine-coarse aggregate ratio and maximum aggregate size varied, it appears from Fig. 16 that the cylinder-cube ratio was not only a function of strength, but also of the mix design.

Due to poor compaction and curing, the characteristic in situ strength may be lower than the strength of corresponding laboratory test specimens. For normal-strength concrete, various codes assume a factor in the range of 0,75 to 0,90. To provide some information on high-strength concrete, an extensive test programme was carried out on three Norwegian offshore concrete platforms, the results of which are shown in Fig. 17.

The long-term sustained strength of concrete is less than that of short-term loading. For normal-strength concrete a sustained strength of about 70 to 80% of the ultimate short-term strength is often observed, while for high-strength concrete a somewhat higher strength of 80 to 85% has been observed(59).

It should be noted that the strength development rate of high-strength concrete beyond 28 days is generally lower than that of normal-strength concrete, which may be due to lack of water for further hydration(60,61).

For dense concrete with silica fume which has been cured in laboratory air over a long period of time, a slight reduction in long-term strength has been observed(62). Such observations may be explained by stresses being induced by inhomogenous drying, in such a way that tension builds up in the surface layer of the concrete specimens, while compression is created in the core(63). Other investigations on long-term strength carried out both on existing concrete structures and laboratory specimens with grade and age of the concrete varying from 40 to 165 MPa and 2 to 10 years, respectively, have revealed no tendency towards the loss of strength(64-66).

Tensile and flexural strengths

Although the tensile strength generally increases with increasing compressive strength, the increase is not directly proportional to the compressive strength. According to ACI Committee 363(5) the splitting tensile strength of normal-strength concrete may be as high as 10% of the compressive strength, but at higher strengths of up to about 85 MPa it may be as low as 5%. The splitting strength is higher for crushed aggregate compared to that of gravel aggregate.

The following equation for the prediction of splitting tensile strength has been suggested by Carasquillo et al.(67):

$$f_{sp} = 0{,}54\sqrt{f_{cc}}\ \text{MPa} \tag{2}$$

The equation is shown in Fig. 18.

In the Norwegian Concrete Code(1) where the direct tensile strength is used instead of splitting strength, the direct tensile strength is assumed to be about 2/3 of the splitting tensile strength. The following equation for estimating the characteristic direct tensile strength has been given by Thorenfeldt(68):

$$f_{tk} = \underset{3}{0{,}3}\ f_{ck}^{\ 0{,}6} \tag{3}$$

This equation is shown in Fig. 19.

Another typical tensile strength parameter is the flexural strength. ACI Committee 363(5) reports the flexural strength to be about 1,4 to 1,6 times the splitting tensile strength, both for normal and high-strength concrete.

Short-term deformation

As shown in Fig. 20 the ascending part of the stress-strain diagram for high-strength concrete becomes more steep and linear up to a higher percent of the ultimate stress than that of normal-strength concrete. The strain at peak load increases slightly with strength, and the decending part of the diagram also becomes steeper.

The shape of the stress-strain diagram is closely related to the nature of concrete as a composite material. Hence, the elastic properties of the aggregate and the aggregate-cement paste bond also very much affect the stress-strain behavior of the concrete. The improvement of the aggregate-cement paste transition zone further affects the hysteresis loop and the unrecoverable strain after unloading in such a way that the width of the hysteresis loop and the unrecoverable strain are reduced as the strength of the transition zone is increased(14).

Based on a number of investigations(67,70-72) it seems that both ACI 318-89 and Model Code MC-90 overestimate the modulus of elasticity for concrete of compressive strengths above 40-50 MPa. The Norwegian Concrete Code(1) gives the following equation:

$$E_c = 9.500\, f_{cc}^{0,3} \qquad (4)$$

for $f_c < 85$ MPa.

Carrasquillo et al. (67) have given:

$$E_c = 3320\sqrt{f_{cc}} + 6.900 \text{ MPa} \qquad (5)$$

for $f_c < 83$ MPa

Both of these expressions, together with some test results are shown in Fig. 21.

Long-term deformation

Not much information on long-term deformation on high-strength concrete is available. For concrete with very low w/c ratios, however, antogenous shrinkage due to self-dessication may represent an additional deformation which needs attention in addition to drying shrinkage and creep.

As can be seen from Table 4 it is primarily the presence of silica fume in high-strength concrete, which appears to increase the autogenous shrinkage. While high-strength concrete without silica fume has an autogenous shrinkage of the same order as that of normal-strength concrete, the presence of silica fume may increase the autogenous shrinkage by up to twice that of normal-strength concrete.

For concrete with high w/c ratios, it is the drying shrinkage which is dominating due to the successive drying out of free water from the concrete. For concrete with very low w/c ratios, however, the free water is primarily consumed by cement hydration. If silica fume is present, the microstructure is further changed towards finer pores in such a way that surface tension is increased and consequently also the autogenous shrinkage.

Normally, the total shrinkage is observed in the form of a combined autogenous and drying shrinkage. Whether the total shrinkage of high-strength concrete is lower or higher than that of normal-strength concrete, depends on local drying and curing conditions. de Larrard et al.(76) have shown, however, that the autogenous shrinkage of high-strength concrete can be reduced by replacing a certain amount of the cement by a calcareous filler.

As far as creep is concerned, it has been shown that the basic creep is directly related to the presence of free water in the cement paste, and that dry concrete does not exhibit any creep(77). When the w/c ratio decreases, both the quantity of free water and the volume of hydrates are reduced. Thus, for constant paste volume, a reduced basic creep should be expected, which is also supported by experimental results (Fig. 22).

Not too much information is available on drying creep of high-strength concrete. Experimental results indicate, however, that the drying creep of high-strength concrete is reduced compared to that of normal-strength concrete (Fig. 23). In Fig. 24 the specific creep is plotted versus concrete strength from data reported in different investigations.

DURABILITY

Electro-chemical corrosion

Since high-strength concrete is characterized by a low porosity and a more uniform microstructure compared to that of normal-strength concrete, this indicates a high resistance to penetration of carbon dioxide and chloride ions into the concrete. However, during production of high-strength concrete, both macrocracking due to plastic shrinkage and microcracking due to self-dessication may represent potential problems from a corrosion protection point of view. Also, somewhat depending on the amount of mineral admixture used both the reserve basisity and the ability of the cement paste to bind chlorides may be affected.

By replacing part of the portland cement with a mineral admixture such as fly ash, only modest effects on the pore solution chemistry have been observed(80), but this may vary with the type of fly ash. Unlike fly ash, however, the presence of silica fume has an extensive and profound effect on the pore solution chemistry(80,81).

For an increasing replacement of the cement by silica fume, the concentrations of both K^+ and OH^- ions are substantially lowered. However, by replacing up to 20% it appears from Fig. 25 that the pH does not drop below that of a saturated calcium hydroxide solution which is approximately 12.5. Even at 30% cement replacement the pH does not drop below 11,5, which is considered to be a threshold value for maintaining a good passivity of embedded steel. According to Diamond(80) it appears that the removal of alkalies from the pore solution with consequent lowering of pH is less complete at low w/c ratios. Thus, it appears that it is carbonation of the concrete and penetration of chlorides which should be the controlling factors for the passivity of embedded steel. There are no data reported in the literature, however, that carbonation of high-strength concrete either with or without mineral admixtures represents any problem.

Nakumara et al.(82) have investigated chloride binding of slag mortars with w/c = 0,40 using alternative 24 hr. periods of immersing in seawater and oven drying at 60°C. The specimens were then analysed for chlorides at three levels, the test results of which are given in Table 5. The high surface chloride content of the ordinary slag specimens is attributed to the greater binding capacity for chlorides in this blend than in the ordinary portland cement. The penetration to the middle layer suggests that there is some beneficial effect of the slag, the greatest being for the very finely ground slag with additions of anhydrite. The chloride contents of the inner layers were of the same order as of the reference specimens without exposure to chlorides. It should be noted, however, that these were immature specimens; they were only 7 days old at the beginning of the testing which lasted for only a further 14 days. A greater effect of the slag would be expected for older specimens.

When silica fume is used as a partial replacement for the cement, there is not yet a general consencus of the influence of chloride-binding. Page and Vennesland(81) determined the chloride binding capacity to be substantially reduced as shown in Fig. 26, wheras Byfors et al.(83) observed an increase in the amount of bond Cl^- in ordinary portland cement with 10 % silica fume additions relative to the ordinary portland cement. If the concrete becomes carbonated, it is well established that the capacity for chloride binding is distinctly reduced(84). Since carbonation is not a problem for high-strength concrete, the effect of carbonation on a possible chloride penetration should not represent any problem.

Recent investigations on offshore concrete platforms in the North Sea indicate that the rate of chloride penetration into concrete made with portland cement alone is much higher than expected earlier(11), but not much information on chloride penetration through high-strength concrete is available. Since the rate of chloride penetration is generally reduced with reduced permeability, high-strength concrete should provide a better protection against chloride penetration than normal concrete. Addition of mineral admixtures such as silica fume, slag or fly ash also increases the resistance to chloride penetration(85). If sufficient amounts of chloride reach the embedded steel, however, electrical resistivity and availability of oxygen are the additional factors controlling the corrosion rate.

Since an electrical current passes through the concrete in the form of charged ions, it is reasonable to assume that there is a close relationship between electrical resistivity, ion concentration, and porosity. The moisture content of concrete is also an important factor. If the concrete is dry enough, the resistivity may be too high to allow any significant transport of ions, and significant corrosion rate will not occur.

Fig. 27 illustrates the effect of moisture content on the electrical resistivity of normal concrete. By successively reducing the moisture content from 100 to 20%, it can be seen that the resistivity increases by approximately three orders of magnitude. In high-strength concrete, the w/c ratio is less than what is theoretically neccessary for complete hydration, where self-dessication of the concrete is a real possibility. Consequently, the mixing water in high-strength concrete will be used up relatively rapidly and long before the clinker hydration is complete. Excluding any interaction with the atmosphere, the relative humidity inside high-strength concrete will be lower than that in normal-strength concrete. If silica fume is also used in the concrete, both the reduced permeability and the changed pore solution chemistry will affect the concrete in such a way that a dramatic increase in electrical resistivity may be observed, in particular at high cement contents (Fig. 28).

For high-strength concrete resistivities of up to 100 kohm cm even for water-saturated conditions have been observed(86). Observations from existing concrete structures indicate that corrosion of embedded steel hardly represents a practical problem if the electrical resistivity exceeds a threshold level of 50 to 70 kohm cm(89). More recent information indicates that resistivities as low as 20 kohm cm can reduce the rate of corrosion to a negligible level(90). Therefore, it appears that high-strength concrete typically will have an electrical resistivity which is above the level where corrosion of embedded steel will represent any practical problem. If the steel should become depassivated, the rate of corrosion will also be controlled by the oxygen availability.

Even for normal concrete very few investigations on oxygen availability have been reported, and for high-strength concrete, no particular information is available. Since the permeability of high-strength concrete is very low, the rate of oxygen transport through high-strength concrete must also be very low. For a cathodic reaction to take place, enough moisture must also be available in order to dissolve the oxygen. For a homogeneous high-strength concrete without any cracks, it is reasonable to assume, therefore, that the oxygen availability will generally be very low.

Although the risk of corrosion is generally higher in cracked concrete than in un-cracked concrete, the only information available on effect of cracks is based on normal concrete. Even with the comprehensive and realistic experiments carried out by Houston et al.(91) on normal concrete, however, it was difficult to establish a simple relationship between crack-width and risk of corrosion. When the concrete elements were investigated at an early stage of exposure, there appeared to be a distinct effect of crack width on corrosion. After longer periods of exposure, however, the effect of crack width was very small or almost non-existent. This is also demonstrated in Fig. 29, which shows results obtained for beams with 25 mm concrete cover after 10 years of exposure in different types of atmospheric environments. Also for submerged concrete, recent information has shown that the risk of cracks on corrosion is less than previously expected(93).

As far as the ability of high-strength concrete to protect embedded steel from corrosion is concerned, not much systematic research has been carried out so far. Based on the general information available, however, it appears that high-strength concrete has a high ability to protect embedded steel. This general conclusion is supported by the good performance observed for offshore concrete platforms in the North Sea, where concrete with compressive strengths of 45 to 70 MPa has been used. For these structures, no problems due to corrosion of embedded steel has been reported so far, even after 15 to 20 years of the combined exposure to heavy mechanical loading and a severe marine environment.

Frost resistance

Even for normal-strength concrete, production of concrete with a good and stable air-void system is normally a problem(94,95), but in the presence of a high dosage of superplasticizer, the establishment of a good and stable air-void system is even a bigger problem(19). For production of high-strength concrete, it may also be a conflicting requirement to entrain air which will decrease the strength. Therefore, much attention has been given to find out whether a frost resistant high-strength concrete can be produced without any air entrainment.

The general problem of assessing the frost resistance of concrete is the lack of correlation between existing laboratory test methods and field performance. Also, different test methods and different test conditions appear to give different and conflicting test results.

In 1981, Okada et al.(96) reported very good frost resistance (ASTM C666A-A) of non-air-entrained high-strength concrete with w/c ratios in the range of 0.25 to 0.35. Similar observations were later on reported by Foy et al.(97), and Gagné et al,(98) who observed good frost resistance of non-air-entrained concrete with w/c ratios of 0,25 and 0,30, respectively.

Using ASTM C666-A Malhotra et al.(99) also tested a number of concretes with different types of cement and w/c ratios of 0,30 and 0,35. They concluded, however, that air entrainment was necessary for these concretes to be frost resistant. Hammer and Sellevold(100) tested non-air-entrained concrete with 0 and 10% silica fume and w/c ratios varying from 0,25 to 0,40. Even for the lowest w/c ratios some of the specimens were damaged during testing. These observations were in conflict with low-temperature calorimeter data, which clearly demonstrated a very low freezable water content. Based upon this, Hammer and Sellevold(100) suggested, therefore, that the observed damage could be due to thermal fatique caused by too large differences between the thermal expansion coefficients of aggregate and binder rather than ice formation.

As far as salt scaling is concerned, there are also some conflicting results reported in the literature. In 1984 Petersson(101) reported that deterioration of high-strength concrete due to salt scaling was small for the first 50 to 100 cycles, but increased very rapidly to total destruction in the following 10 to 20 cycles. Foy et al.(97), however, observed that the resistance to salt scaling of concrete with a w/c ratio of 0,25 was very good even after 150 cycles. Both Hammer and Sellevold(100) and Gagné et al.(98) have demonstrated that it is possible to produce high-strength concrete which is resistant to salt scaling without any air entrainment. These test results included w/c ratios of up to 0,37.

Chemical resistance

According to Biczók(102) chemical deterioration of concrete can be classified into three types of processes depending on the predominant chemical reaction taking place. Leaching corrosion of concrete is the process where parts or all of the hardened cement paste are removed from the concrete. Normally, this is caused by the action of water of low carbonate hardness or carbonic acid content. The next process is corrosion by exchange reactions and by removal of readily soluble compounds from the hardened cement paste. This process occurs as a result of a base exchange reaction between the readily soluble compounds of hardened cement paste and the aggressive solution. The third process is the swelling corrosion, largely due to the formation of new and stable compounds in the hardened cement paste. This process is primarily the result of attack by certain salts. Also alkali-aggregate reactions cause expansion, where the concrete eventually is destroyed by a swelling pressure.

For all of the above deteriorating processes, the permeability of the concrete is the key factor governing the rate of deterioration (Fig. 30). In addition, calcium hydroxide is also an easily soluble constituent which is very vulnerable to chemical attack.

In sulfate-containing solutions the calcium hydroxide reacts with the sulfates to produce gypsum which may further react with the aluminates to form ettringite. Both gypsum and ettringite can cause disruptive expansion. Therefore, the pozzolans normally used in high-strength concrete are very effective in reducing the calcium hydroxide (Fig. 12), and hence also greatly enhancing the resistance against sulfate attack. In addition to reducing the calcium hydroxide, pozzolans, such as silica fume, also form calcium silicate hydrates which are able to incorporate aluminum ions, thus reducing the amount of aluminum alvailable for ettringite formation.

The beneficial effect of silica fume in high sulfate-containing environments has been reported from several investigations(104,105). In these investigations, the performance achieved has been equal or better than that obtained by use of sulfate resistant cements. Mehta(106) exposed a number of high-strength concretes to solutions of both 5% sodium sulfate and 1% concentrations of sulfuric and hydrochloric acid for a period of up to 182 days. Although the portland cement contained 7% C_3A, the results showed that w/c ratios of 0,33 to 0,35 gave a too low permeability to cause any deterioration. In more aggressive environments pure portland cements have shown some deterioration, wheras additions of silica fume have given practically unaffected performance(105). Even for normal-strength concrete the presence of silica fume will improve the long-term performance of concrete in very aggressive environments. In 1952 a large number of concrete specimens were exposed in a field station in the underground of Oslo city, which consists of very aggressive alum shale. In spite of a sulfate content of up to 5 g/l of SO_3 and a pH of 2,8, a 15% replacement of ordinary portland cement with silica fume gave the same good performance after 26 years of exposure as that by a sulfate resisting cements (ASTM Type V)(107).

The presence of pozzolans such as silica fume can also be used to control the expansion caused by alkali-aggregate reaction(108,109). Porewater analyses of cement paste with silica fume have demonstrated the ability of silica fume to reduce the alkali concentration in the pore water rapidly, thus making it unavailable for the slower reaction with reactive silica in the aggregate(80,81).

Asgeirsson and Gudmundsson(110) used silica fume with Icelandic high-alkali cement and reactive sand in mortar bars to demonstrate the ability of silica fume to reduce the expansion. Later on, field experience with Icelandic cement blended with silica fume with 200 houses constructed during the period of 1979-86 has confirmed these observations(111).

Fire resistance

Diederichs et al.(112) have shown that high-strength concrete is more vulnerable to elevated temperautres compared to that of normal-strength concrete (Fig. 31). For high-strength concrete a distinct loss of strength (30%) was observed already at 150°C, while normal concrete retained its strength up to 350°C. The effects on compressive strength and modulus of elasticity are shown in Figs 32 and 33, respectively.

Investigations carried out on the fire resistance of high-strength concrete are rather limited. In 1981 Pedersen(113) reported the results of an investigation where 100 mm cylinders of high-strength concrete with a w/c ratio of 0, 16 and 20% silica fume were tested. During heating up at a rate of 1°C/min, several of the specimens suddenly disintegrated at a temperature of approximately 300°C. Shirley et al.(114), however, tested concrete slabs according to ASTM E119(115), where five different types of concrete with strengths varying from 50 to 120 MPa revealed no significant difference in behavior. There was no explosive behavior, and none of the concretes showed even minor spalling on the exposed surface. Jensen et al.(116) reported a number of test series where different types of concrete elements were exposed to a hydrocarbon fire, where 1100°C is reached in 30 min. The strength of the elements varied from 40 to 80MPa. The test results showed that spalling and damage occurred earlier than expected and that increasing moisture content increased the severity of the effects. Concrete with silica fume was also more sensitive to spalling. Williamson(117) tested concrete slabs according to ASTM 119 with two reference mixes (30 and 80 MPa) and two mixes with silica fume (50 and 100 MPa). The relative humidity in the concrete was monitored during the storage period of about six months, where a level of 71-74% RH was reached at the time of testing. No external spalling or internal damage to any of the specimens was observed.

Although the above test results appear to be contradictory, it seems that increasing moisture content leads to increased damage. Also, concrete with silica fume is more dense and dries out more slowly in such a way that a higher vapour pressure can build up internally and cause spalling(118).

Abrasion resistance

Although some concrete codes have recently increased their upper strength level for structural utilization of concrete to about 100 MPa(1), very few structures have so far been built with concrete strengths of more than 70 MPa. At the same time, mineral aggregate-based concrete with very high strengths of up to 230 MPa can now be produced (2). There appears, therefore, to be a great potential for utilization of high-strength concrete to applications where high abrasion resistance is needed.

Mechanical abrasion is the dominant abrasion for pavements and bridge decks exposed to studded tires. This effect has been studied under controlled laboratory conditions; an accelerated load facility for full-scale testig of abrasion resistance of highway pavements exposed to heavy traffic by studded tires was built in Norway in 1985 (Fig. 34). Fig. 35 presents some data from that test facility. By increasing the concrete strength from 50 up to 100 MPa the abrasion of the concrete was reduced by roughly 50%. At 150 MPa the abrasion of the concrete was reduced to the same low level as that of high quality massive granite. Compared to an Ab 16t type of asphalt for a typical Norwegian highway traffic, this represents an increased sevice life of the highway pavement by a factor of approximately 10.

In the Scandinavian countries where studded tires are extensively used, even high quality asphalt pavements may have a service life of only two to three years or less. Encourged by the above test results several high-strength concrete pavements have been completed over recent years(120).

The wear of non-pavement surfaces such as concrete floors, sidewalks, stairs, etc. is caused primarily by foot traffic, light vehicular traffic, and the skidding, scraping, or sliding of objects on the surface. In some types of industrial operations, the use of steel wheel vehicles, forks, buckets of lift trucks and loaders inflicts very severe damage to the concrete surfaces they operate on. The movement of abrasive granular material in and out of concrete storage facilities such as silos and bins also creates significant abrasion problems. In the Scandinavian countries the utilization of high-strength concrete for such problems has already started.

Hydraulic abrasion, or abrasion erosion is recognizable by the smooth, worn concrete surface in addition to cavitation erosion, where the surface is full of small holes and pits. Spillway aprons, stilling basins, sluiceways and tunnel linings are particularly susceptible to abrasion erosion.

Most concretes used in hydraulic structures in the past could not be classified as high-strength concrete. The concrete provided the mass, and strength was a secondary consideration. Concerns about thermal cracking at early ages in the large concrete sections used in these structures have led to low cement contents to minimize heat of hydration, with subsequent low concrete strengths. Where higher quality concrete is used, the resistance to high water velocities has been satisfactory for many years, but even these concretes do not fully resist the abrasive action, grinding or repeating impacts of the debris. Observations of abrasion-erosion of concrete surface in the stilling basins of several major USA dams have varied from 50 mm to 3000 mm. At Dworshak Dam, 1530 m^3 of concrete and bedrock were eroded from the stilling basin. In many of these instances, the abrasion-erosion is accelerated because of the impact forces of large rocks and boulders caught in turbulent flows. These forces weaken the concrete surface and make it more susceptible to removal.

In 1983 a major repair on the stilling basin of Kinzua Dam in Pennsylvania, USA, was carried out based on high-strength concrete(121). The structure which has been in operation since 1967, had already had an extensive repair carried out in 1973-74. In 1983 approximately 1500 m^3 of 250 mm slump concrete was placed, the 28 day compressive strength of which was 90 MPa. Diver inspection of the concrete after one year of service including a period with very large volume of debris in the stilling basin, showed that the concrete was performing as intended.

For marine or offshore concrete structures exposed to "ice abrasion", the actual mechanism that results in loss of surface is more complex than the simple act of ice rubbing on the concrete surface. Research work and field observations suggest that the concrete deterioration at or near the waterline is due to a combination of environmental causes plus the impact loading of the concrete surface by repeated impacts from the ice floes.

Pieces of ice, driven by wind and current, can possess significant kinetic energy, much of which is dissipated into the concrete when the ice collides with the concrete structure. Some of the energy is lost in the crushing of the ice. The frequency of the loading is dependent on the circumstances which occur at any given time at a particular structure and can vary from an occasional impact to repeated impacts every few seconds. A large ice floe in open waters will, upon initial contact with a structure, both load the structure and begin to crush itself at the point of contact with the structure. As the driving forces of the floe continue to move it forward against the structure, the resistance of the structure continues to increase to a point where the floe experiences a local failure in the ice, usually in flexure, some distance from the initial point of contact with the structure. This momentarily releases the load on the structure. The original ice, now damaged by crushing and cracking, is shunted away by the moving flow and new, undamaged ice in the floe then collides with the structure. The characteristics of the ice and the floe, combined with the dynamic response of the structure, will establish a "ratcheting" effect on the concrete surface, repeatedly loading and unloading it. With time, this repetitive loading behavior can destroy the effectiveness of the aggregate bond near the surface of the concrete and both cause and propagate existing microcracks in the concrete.

Regardless of type of abrasion, both laboratory and field experience indicate that compressive strength is the single most important factor in determining the abrasion resistance of the concrete. Also, the abrasion resistance can be significantly improved by the use of hard and dense aggregate both in the upper and lower part of the grading curve. By replacing the 2 to 4 mm fraction of the natural sand with crushed high quality material, the compressive strength in the Norwegian highway pavement investigations(119) decreased from 164,3 to 153,3 MPa, while the service life of the pavement increased by 50%. Generally, the abrasion of concrete is higher in wet than in dry condition, but the experience indicates that also this effect is reduced by increased strength level. From Fig. 35 it can be seen that at 50 MPa, the wet abrasion loss was approximately 100% higher than the dry abrasion loss, while at 100 MPa the wet abrasion loss was only 50% higher. At 150 MPa, only a small difference between the wet and dry abrasion loss was observed. For such a dense concrete, it appears that the effect of moisture becomes more negligible.

CONCLUDING REMARKS

Recent developments in concrete technology have made it possible to produce concrete mixtures with strength properties that are far beyond what is possible to utilize by the current structural design practice. Very often it is not the improved strength which is the primary objective but rather the overall improved performance. Therefore, the term "high-performance concrete" covers "high-strength concrete".

There are several reasons for the above development. A rapid development in the general field of materials science has taken place. New cementitious materials and admixtures have been developed, and advancement in processing of aggregates have also taken place. Reinforced and prestressed concrete is being utilized in new areas, such as offshore structures for oil and gas explorations. Thus, from the first concrete structure produced for the North Sea operations in 1973 (the Ekofisk tank) to the most recent offshore structures currently under design, the quality of concrete has increased from 40 MPa to 80 MPa. In North America high-strength concrete is being used in tall buildings, where up to 130 MPa concrete has been used in the columns(122).

For highway pavements and bridge decks subjected to studded tires, experimental work has shown that utilization of high-strength concrete may increase the service life by a factor of up to 10, compared to that of a high-quality asphalt pavement. Industrial floors, hydraulic structures, and structures exposed to ice abrasion also represent a great potential for utilization of high-strength concrete. In many areas, large quantities of resources are being spent on maintenance and rehabilitation of concrete structures due to lack of durability. There is a great challenge, therefore, for the engineering profession to utilize and further develop the technology of high-strength concrete or high-performance concrete for the benefit of the society.

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Table 1. Effect of silica fume on the calcium/silicate-ratio of the hydrates(38).

Cement	Ca/Si-Ratio
Portland cement (OPC)	1.6
OPC + 13% silica fume	1.3
OPC + 28% silica fume	0.9

Table 2. Conversion factor between different cube and cylinder specimens(56).

	Cube (mm)			Cylinder (mm)
Specimen	100	150	200	150/300
Cube 100	1	0.99	0.95	0.82
Cube 150	-	1	0.96	0.83
Cube 200	-	-	1	0.87

Table 3. Conversion factor between cube and different cylinder specimens(57).

100 mm Cube MPa	100 x 300 mm Cylinder strength cyl./cube	150 x 300 mm Cylinder strength cyl./cube
66.3	0.73	0.75
79.7	0.73	-
97.0	0.77	0.77
115.4	0.82	0.83

Table 4. Effect of silica fume on the autogenous shrinkage.

Type of concrete	Autogenous shrinkage µm/m	References
NSC	90 - 120	(73, 74)
HSC	80 - 90	(74, 75)
HSC with silica fume	140 - 290	(73 - 75)

Table 5. Effect of slag on chloride penetration(82).

Type of mix	Mix proportion			Chloride concentration (ppm)		
Material %	W/C	C/S	HRWR %	Surface layer 0-7 mm	Medium layer 7-14 mm	Deep layer 14-20 mm
Control 0	0.4	1/1.5	1.0	3970	521	42
OR slag 40	0.4	1/1.5	1.0	4570	259	62
VF slag 40	0.4	1/1.5	1.0	2910	111	34
VFG slag 40	0.4	1/1.5	1.0	3350	52	28
Silica fume 10	0.4	1/1.5	1.0	5110	312	43

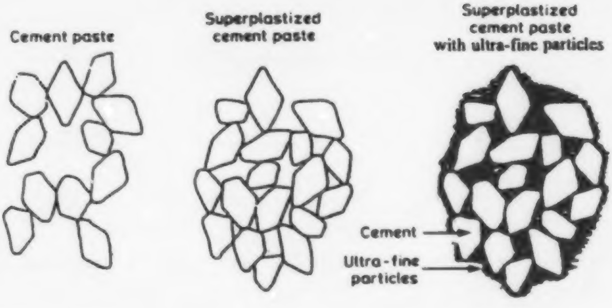

Cement paste
Superplastized cement paste
Superplastized cement paste with ultra-fine particles
Cement
Ultra-fine particles

Fig. 1. Effect o:f superlasticizer and silica fume on the particle packing. (From ref. 20).

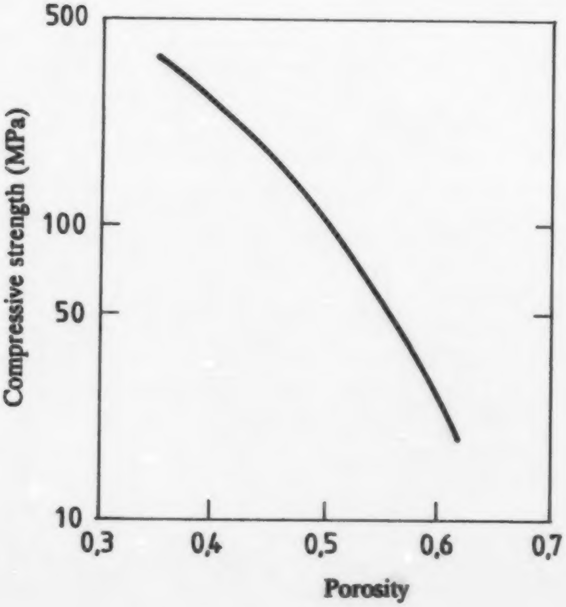


Fig. 2. Effect of porosity on compressive strength of cement paste. (From ref. 20).

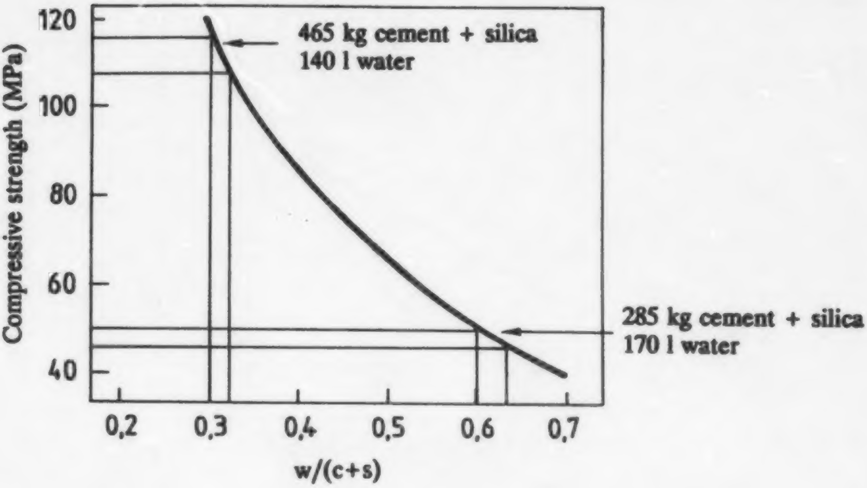


Fig. 3. A variation of 1% moisture content in the fine aggregate will have a much bigger impact on the strength at a very low compared to that at a more normal or high w/c ratio. (From ref. 28).

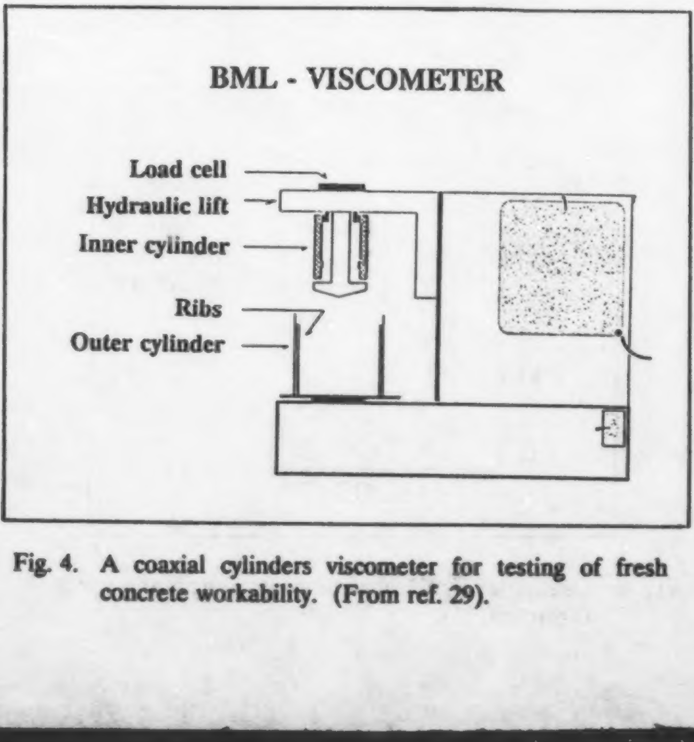

BML - VISCOMETER
Load cell
Hydraulic lift
Inner cylinder
Ribs
Outer cylinder

Fig. 4. A coaxial cylinders viscometer for testing of fresh concrete workability. (From ref. 29).

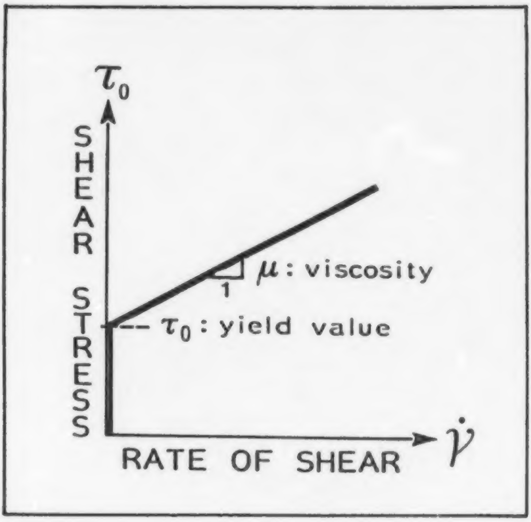


Fig. 5. Bingham behavior.

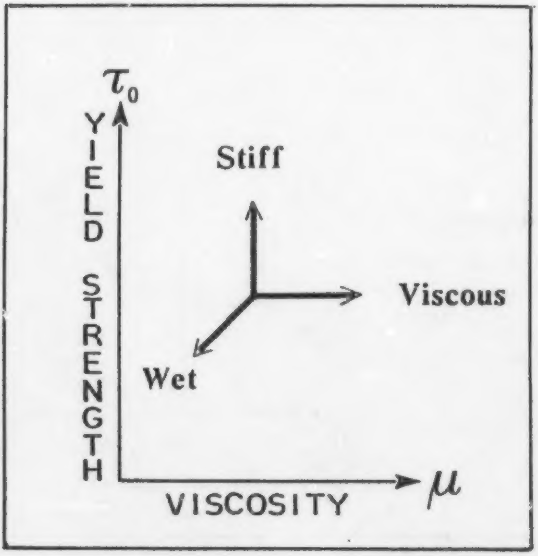

τ_0
YIELD STRENGTH
Stiff
Viscous
Wet
VISCOSITY
μ

Fig. 6. Qualitative significance of yield strength and viscosity. (From ref. 31).

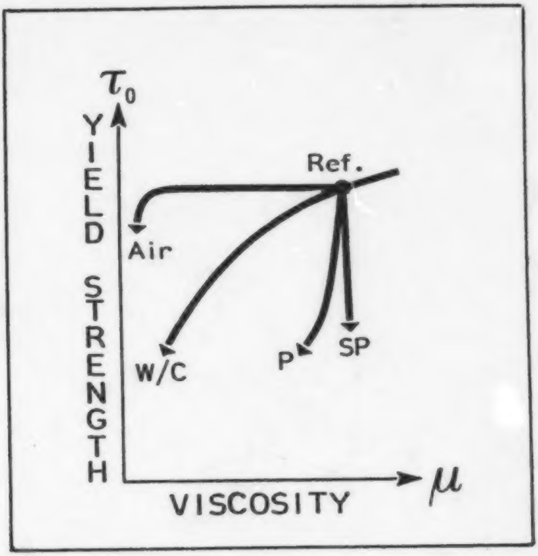


Fig. 7. Effect of various admixtures on yield strength and viscosity. (From ref. 31).

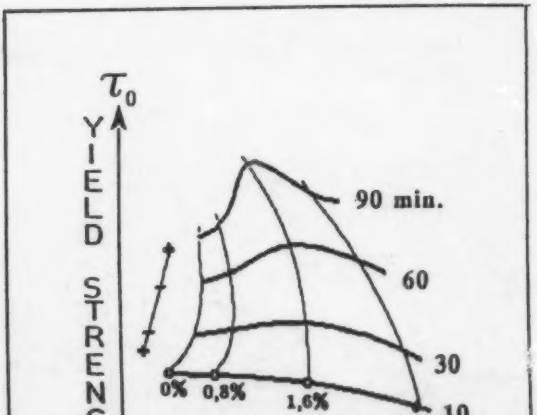


Fig. 8. Effect of superplasticizers and time on yield strength and viscosity. (From ref. 31).

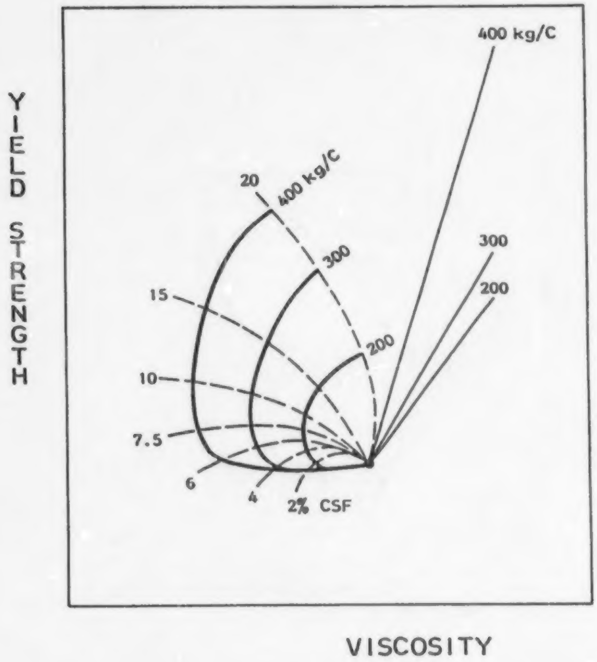
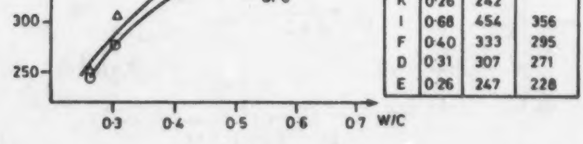


Fig. 9. Effect of silica fume on yield strength and viscosity. (From ref. 31).



Mix	W/C	KJ/kg (C)	KJ/kg (c+s)
J	0·68	380	
H	0·40	330	
G	0·30	278	
K	0·26	242	
I	0·68	454	356
F	0·40	333	295
D	0·31	307	271
E	0·26	247	228

Fig. 10. Effect of w/c ratio on heat of hydration of ordinary portland cement with and without silica fume. (From ref. 34).

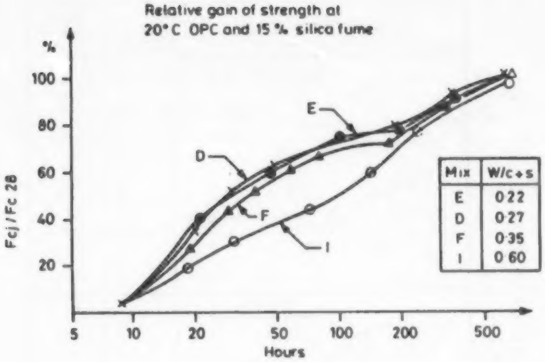


Fig. 11. Effect of w/c ratio on strength development. (From ref. 34).

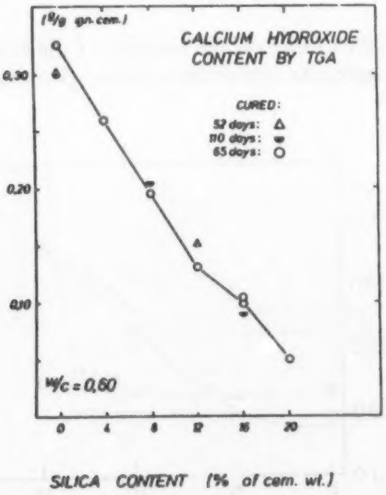


Fig. 12. Effect of silica fume on the calcium hydroxide content of ordinary portland cement paste. (From ref. 37).

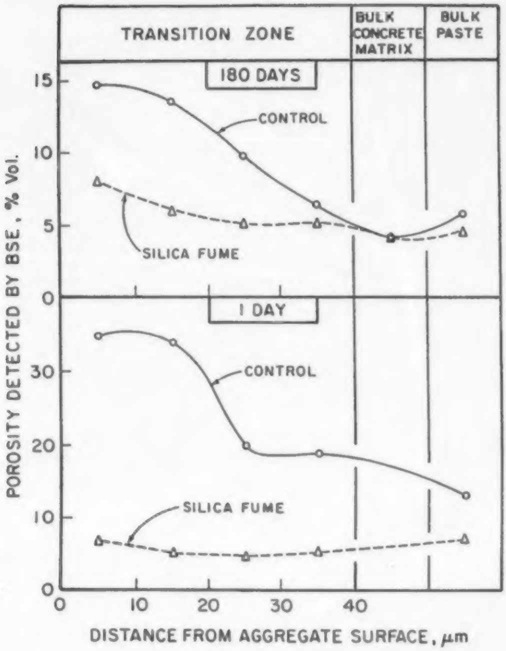


Fig. 13. Effect of silica fume on the porosity of the transition zone between aggregate and cement paste. (From ref. 48).

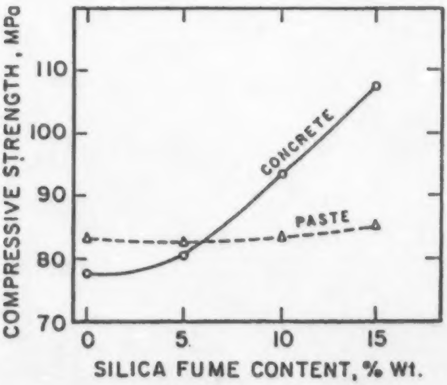


Fig. 14. Effect of silica fume on the strength of 28 days old paste and concrete of the same w/c ratio of 0,33. (From ref. 49).

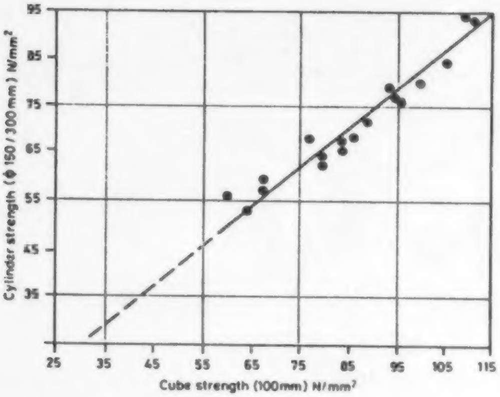


Fig. 15. Relationship between cube (100 mm) and cylinder strength (150 x 300 mm). (From ref. 56).

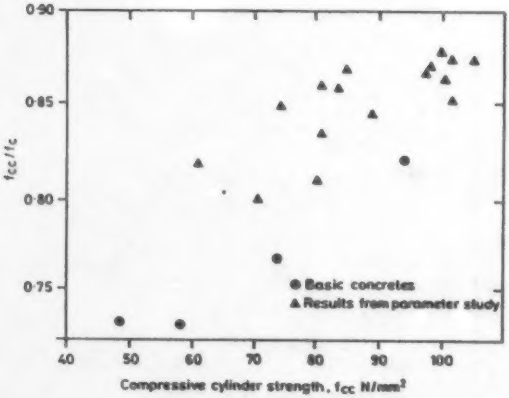


Fig. 16. The conversion factor between cylinder and cube strength as a function of cylinder strength. (From ref. 57).

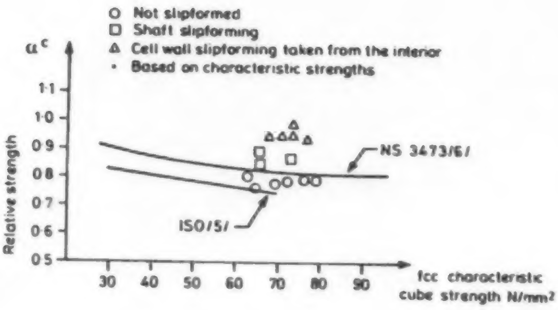


Fig. 17. Obtained in situ strength compared to assumptions in relevant codes. (From ref. 58).

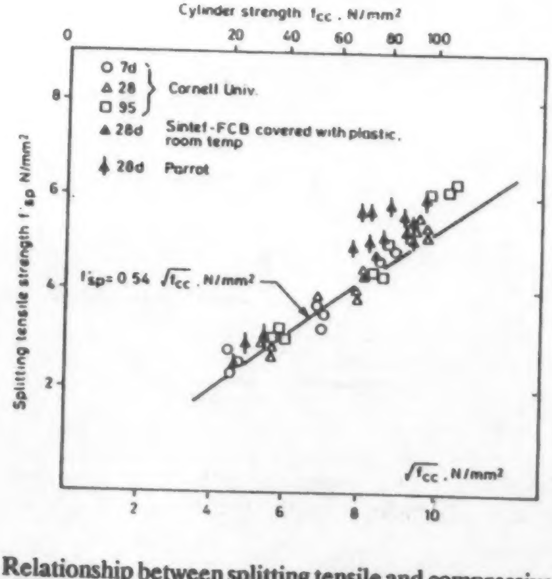


Fig. 18. Relationship between splitting tensile and compressive strength. (From ref. 11).

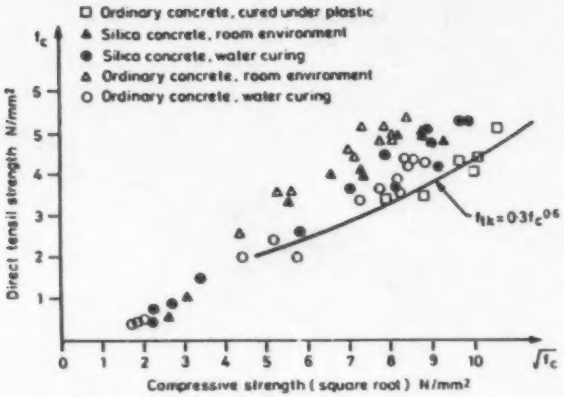


Fig. 19. Relationship between direct tensile and compressive strength. (From ref. 11).

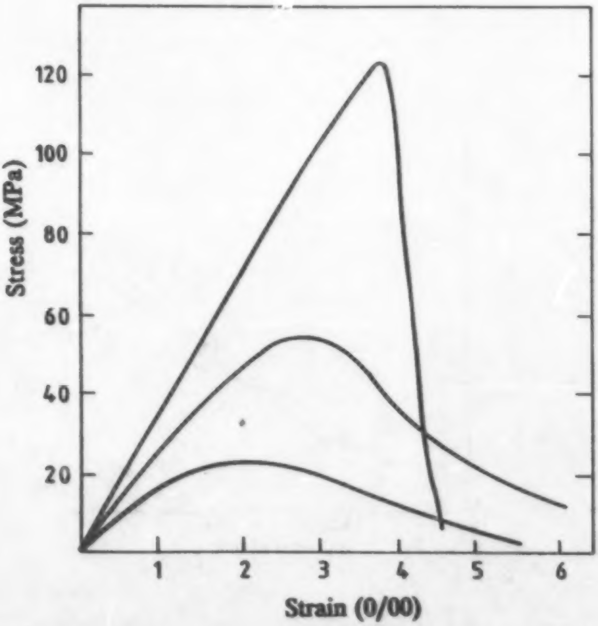


Fig. 20. Stress-strain relationship of high-strength concrete compared to that of normal- and low-strength concrete. (From ref. 69).

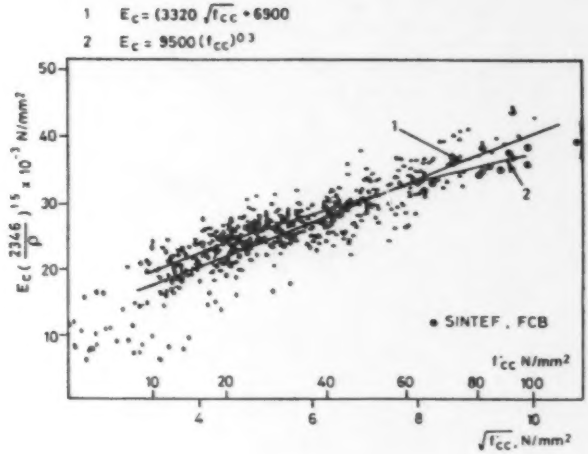


Fig. 21. Relationship between modulus of elasticity and compressive strength. (From ref. 11).

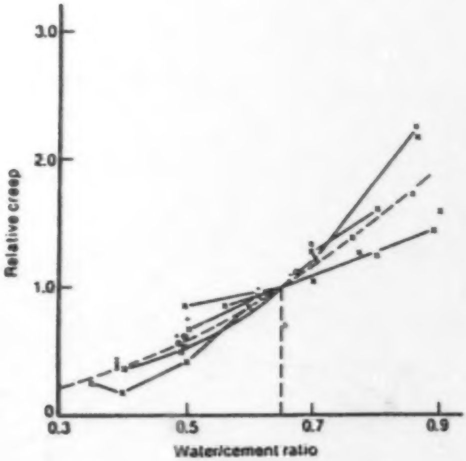


Fig. 22. Effect of w/c ratio on basic creep. (From ref. 78).

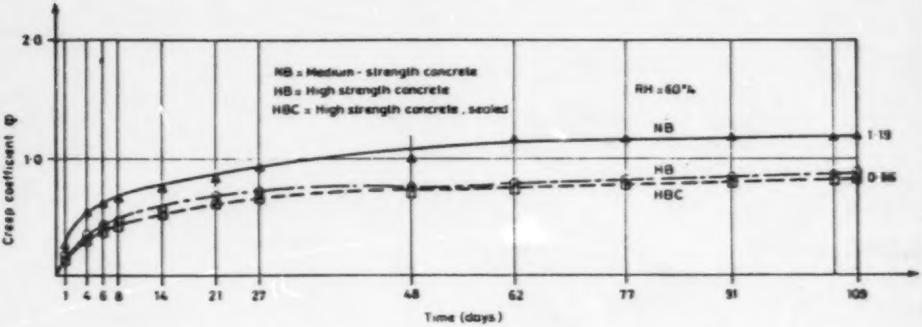


Fig. 23. Creep development for high-strength (80 MPa) and normal-strength concrete. (From ref. 79).

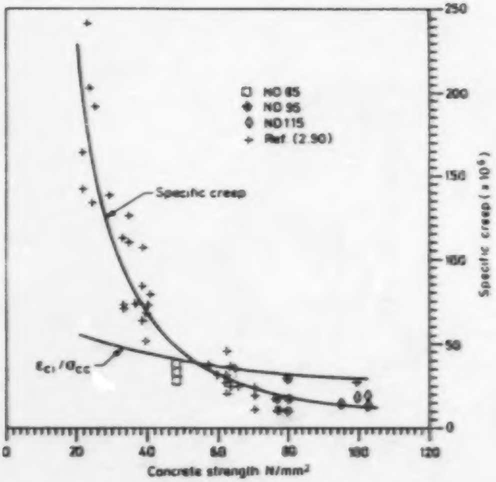


Fig. 24. Relationship between specific creep and compressive strength. (From ref. 11).

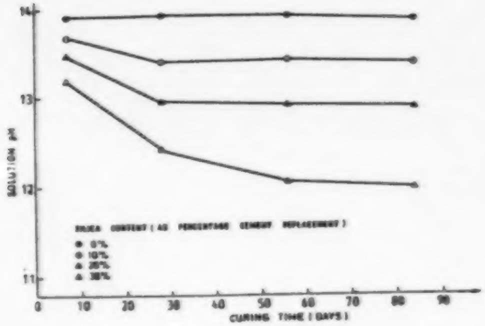


Fig. 25. Effect of silica fume on the alkalinity of cement paste with w/c = 0,50. (From ref. 81).

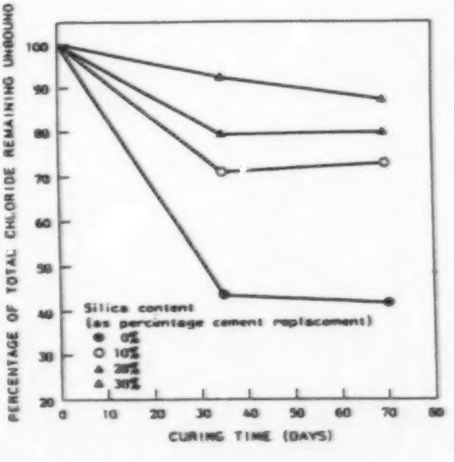


Fig. 26. Effect of silica fume on chloride binding capacity of hydrated cement paste of w/c = 0,50 with 0,40% chloride by weight of cement and silica fume. (From ref. 81).

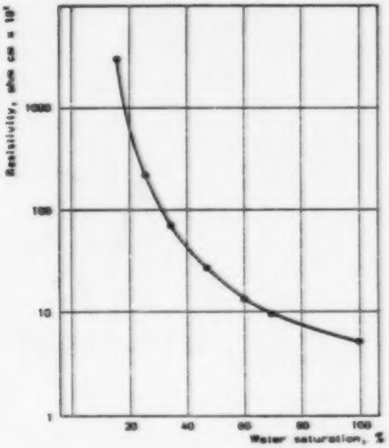


Fig. 27. Effect of moisture content on the electrical resistivity of concrete with w/c = 0,40. (From ref. 86).

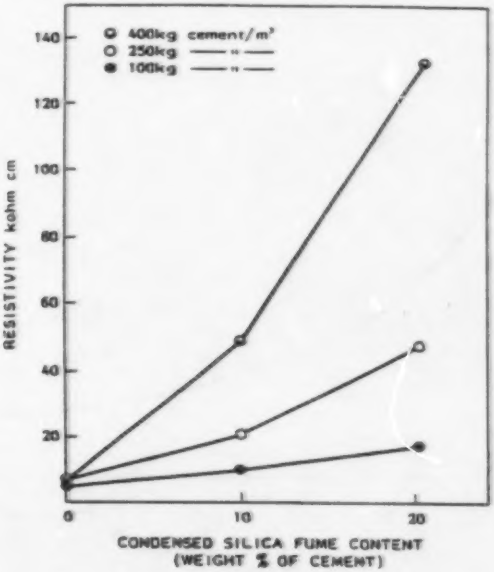


Fig. 28. Effect of silica fume on the electrical resistivity of concrete. (From ref. 87).

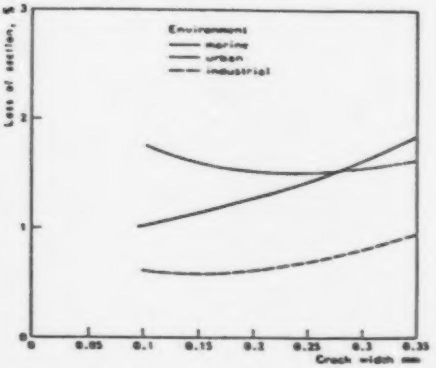


Fig. 29. Effect of crack width on steel corrosion in cracked beams after 10 years of exposure. (From ref. 92).

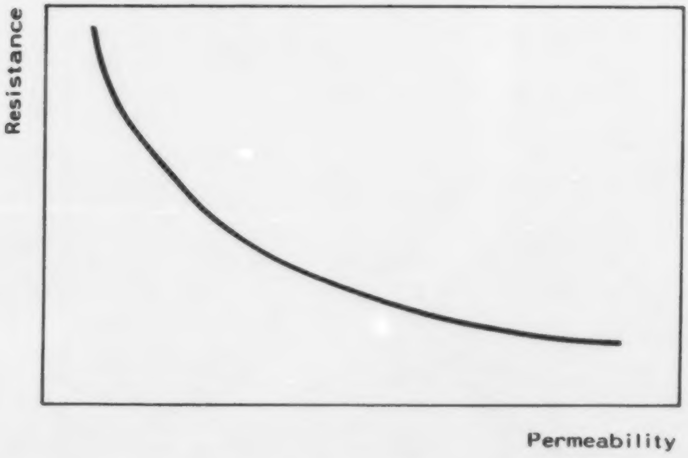


Fig. 30. Relationship between resistance to chemical deterioration and permeability. (From ref. 103).

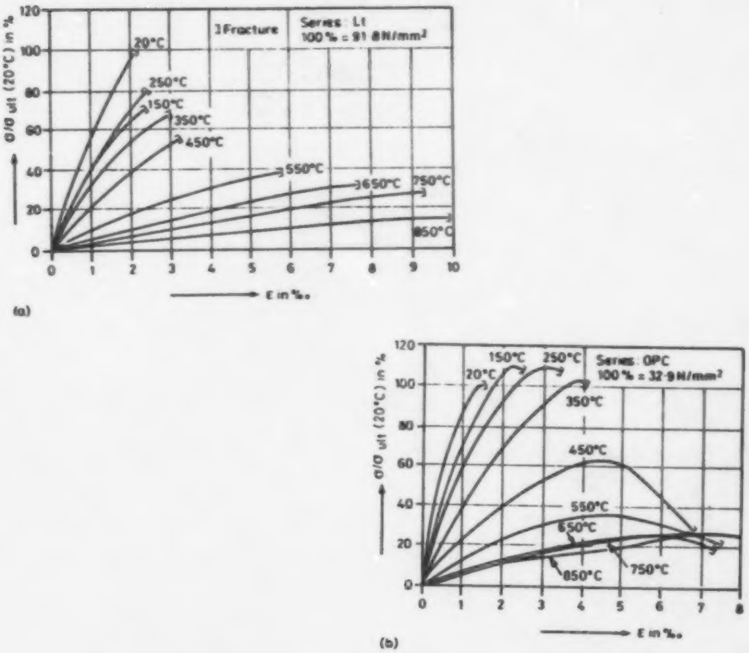


Fig. 31. Effect of elevated temperature on the stress-strain relationships for (a) high-strength concrete with ordinary portland cement and fly ash and (b) ordinary portland cement concrete. (From ref. 112).

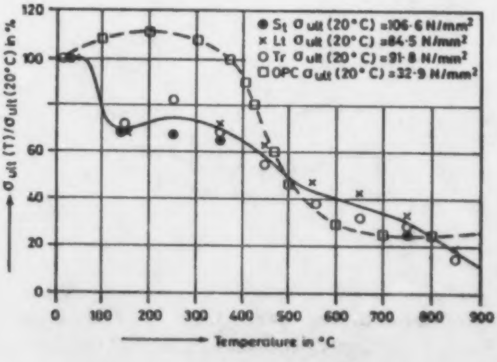


Fig. 32. Effect of elevated temperature on compressive strength. (From ref. 112).

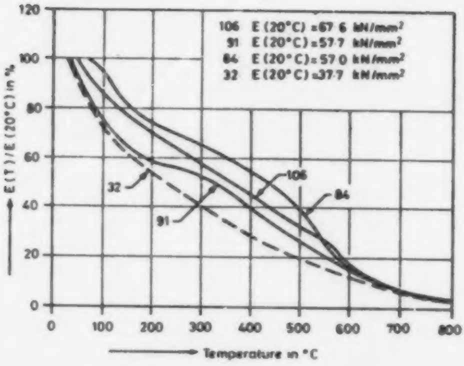


Fig. 33. Effect of elevated temperature on modulus of elasticity. (From ref. 112).

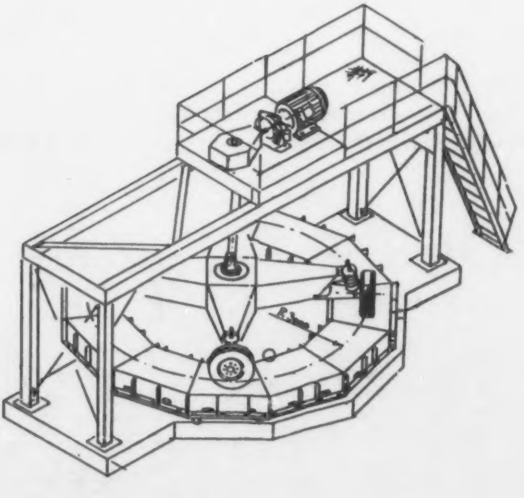


Fig. 34. Accelerated load facility for testing the abrasion resistance of highway pavements exposed to studded tires. (From ref. 119)

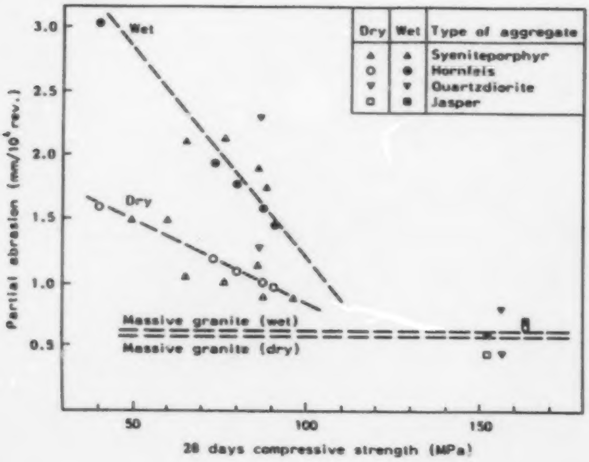


Fig. 35. Relationship between abrasion resistance and compressive strength.

CONCRETE FOR OFFSHORE STRUCTURES

By George C. Hoff

Synopsis: The paper provides background information related to the use of concrete in offshore structures for the development of hydrocarbon resources. Bottom-founded structures, floating structures, and other more specialized structures are described. The use of local concrete materials and labor to produce moderate strength concrete for these platforms is discussed. The various properties of both the unhardened and hardened concrete that are essential in the design and construction of offshore structures are described. Special design and safety considerations are noted. No special construction practices are required. The structures can be built in dry docks, on skidways, or on submersible barges. Portions of the structures can be precast concrete. All of these structures involve some marine operations. The structures require very little maintenance even in the severe marine environment and thus tend to have low life-cycle costs.

Keywords: barge construction, concrete materials, concrete properties, concrete quality, construction practices, design, dry dock construction, floating concrete structures, marine operations, offshore concrete structures, precast concrete, safety, skidway construction.

George C. Hoff received his Doctorate in Civil Engineering from Texas A&M University. He has worked in the field of concrete and construction materials for 30 years and joined the Division of Offshore Engineering of Mobil Oil in 1982. He is the current Chairman of the American Concrete Institute Committee 357, Offshore and Marine Concrete Structures.

INTRODUCTION

Offshore concrete structures are generally understood to be those structures exposed to an open-sea environment (1,2). They are designed to remain permanently or semi-permanently fixed to the sea bed by gravity, piles, or anchors, or to remain afloat and moored. They are often associated with the exploration and production of hydrocarbons but may have many other specialized uses.

Like most other types of concretes, concretes for use in offshore structures are usually made with local materials by local labor in conformance to local guidelines or specifications. Thus, they can vary widely in quality. Depending on their particular application, their strengths can vary from 25 to 65 MPa (3600 to 9500 psi). They are all required to be extremely durable. Once a concrete structure is placed in the sea, maintenance becomes very difficult due to the hostile environment and is very expensive. Some offshore concrete platforms have design lives of 50 to 70 years.

The use of concrete in marine structures goes back to the ancient Romans and Greeks. The use of concrete as a hull construction material for commercial vessels began at the end of the 19th century (3). Initial applications were generally world-wide and consisted of concrete barges and pontoons. The first reinforced concrete sea-going ship was the "Namsenfjord", constructed in Norway in 1917. The first concrete platform for oil and gas production in the Gulf of Mexico was installed in 1950. Since that time, more than 1000 related concrete structures have been built in that area (4) with the first concrete gravity base structure in U.S. waters being installed in 1978 (5). The first large offshore concrete platform for the North Sea (Ekofisk Tank) was installed in 1973. Three concrete platforms, functionally similar to those of the North Sea, have been built in Brazil for South American offshore waters (6). Concrete has a long history and a significant and successful presence in offshore and marine applications.

Throughout the paper, reference will be made to things that are "onshore", "inshore", and "offshore". Onshore is on the land. Inshore means that the location is away from the land but is close enough to the shore to be in protected waters with respect to the open sea. Offshore means that it is located in the open sea. The term "owner" is also frequently used. The owner of an offshore structure can be a single company, or it can be a collection of companies who retain varying percentages of the operation but who have designated a single company to operate and maintain the facility.

TYPES OF CONCRETE STRUCTURES

Offshore structures used in conjunction with hydrocarbon exploration and production can generally be grouped as either being bottom-founded or floating. Many of the bottom-founded structures are also required to float at various stages of their life. The following descriptions of the various types of platforms are very brief but are intended to give the reader a feeling for the enormous versatility that can be realized when concrete is used.

Bottom-founded Structures

Bottom-founded structures can be further identified as:

1. Gravity Base Structures.
2. Concrete Cylinder Pile Supported Structures.
3. Floatable/Bottom-founded Concrete Hull Structures.

Examples of each are shown in Fig. 1, 2 and 3.

The gravity base structure (Fig. 1), commonly called a GBS, maintains its position on the sea bottom due to its very large weight. The sliding force and overturning moment due to the maximum environmental loads are resisted by the weight of the concrete, the operating weights on the structure, and any additional ballast weight that is contained within the structure. This type of structure is common where produced oil must be temporarily stored before being removed to a tanker or pipeline. The practical range of water depths for these platforms is 40 to 350 m (130 to 1150 ft.). These structures are built at onshore or inshore locations and floated out to their final location. They can also be re-floated when platform removal is required (7). More detailed descriptions of these types of platforms can be found in (1), (2) and (8).

Concrete cylinder piled structures (Fig. 2) were the earliest type of concrete offshore platform used. The first Gulf of Mexico platform of this type was installed in 1950. More than 1000 of these platforms have been installed in Lake Maricaibo in Venezuela (4). They consist of an array of prestressed concrete piles which are driven into the seabed. The piles are arranged so that a prefabricated template deck can be placed over the array to form the working surface of the platform. The decks can be made of concrete or any other suitable construction material. Concrete jackets are often placed around the piles in the splash zone and boat impact region of the platform. Steel cross bracing between piles may also be used to stiffen the overall arrangement when the piles become fairly long. The practical range of water depths for these platforms is from 5 to 20 m (16 to 65 ft.). The use of concrete cylinder piles is also common for support of docks, wharves, bridges and roadways over water.

The floatable/bottom-founded concrete hull platforms generally consist of a barge-like concrete hull which is designed to float. Extending upwards from the hull are posts or columns which act as the support frame for the platform (Fig. 3 and 4). These posts or columns can be made of concrete or steel. The hull is floated to its desired location and then water-ballasted down until it sits on the seabed. It is then "pinned" to the seabed by spud piles around its perimeter. These piles maintain the platforms position and help resist sliding and overturning as the platform does not have sufficient on-bottom weight by itself. Once the hull is piled into position, the topsides deck and equipment are usually added using a crane barge. This type of platform has many variations. It can accommodate some subsea storage of produced oil in the hull. The practical range of water depths for these platforms is from 4 to 30 m (13 to 98 ft.). Platforms of this type that are in use in the Gulf of Mexico have, on numerous occasions, been refloated and reused at different locations. Table 1 is a listing of this type of structure constructed by one firm for the Gulf of Mexico and shows typical concrete hull dimensions.

Floating Structures

Floating structures are those structures which will perform their operational function while in a floating mode. These structures will require a permanent mooring system. In general, the current family of floating concrete structures includes:

1. Concrete Tension Leg Platform (TLP).
2. Deep Draft Concrete Floaters (DDCF).
3. Concrete Production/Storage Barges.

Examples of each are shown in Fig. 5, 6 and 7. Large concrete buoy-type floating structures have also been conceptualized.

Concrete Tension Leg Platforms (TLPs) (Fig. 5) derive their name from the fact that they are fastened to large anchors on the seabed by long tethers which have a predetermined amount of tension in them. These tethers, which originate at the corners of the platform, keep the floating platform in a very precise position. The platform itself can have various configurations but generally resembles the semi-submersible drilling rigs which are common throughout the offshore petroleum industry. It consists of an arrangement of base pontoons, shafts or columns which extend upward from the pontoons, and a deck which sits on top of the shafts or columns. The entire hull (pontoons and shafts) and the deck can be made in concrete. The practical range of waterdepth for use of this type of platform is from 300 to 1500 m (1000 to 5000 ft.). The size of the TLP is generally dictated by the amount of operational weight to be carried. Current designs have ranged as high as 50,000 tonnes (55,000 tons).

The Deep Draft Concrete Floater (DDCF) (Fig. 6) is similar in principal to the TLP but uses a conventional mooring system rather than tension tethers. It maintains its positioning during operations due to its extremely deep draft (greater than 130 m (425 ft.)), large weight, low center of gravity, and mooring from the lower portions of the hull. These factors tend to make the structure relatively insensitive to the motions of the sea. Like the TLP, its configuration can have many variations, but in general, it resembles a TLP but with a very deep hull. Similarly, the pontoons, columns, deck, and any bracing can be made in concrete. The practical range of waterdepth for use of a DDCF is from 300 to 900 m (1000 to 3000 ft.). Like the TLP, the size of the DDCF is generally dictated by the amount of operational weight to be carried. Current designs have ranged as high as 50,000 tonnes (55,000 tons).

A variation of the DDCF is the Spar Buoy Platform (Fig. 8). It also takes advantage of the low center of gravity and heavy weight of the concrete to be relatively insensitive to the motion of the sea. It can accommodate crude storage, if desired. It also needs a mooring system (9).

Concrete Production Barges (Fig. 7) are custom-built prestressed concrete barges that provide a support surface for the process equipment, work and storage areas, and living quarters needed for offshore oil and gas production. Drilling is usually not done from these barges but is done from special drilling vessels or jack-up rigs. The production wells are usually located on a nearby unmanned fixed platform. The entire barge or selected portions of the barge can be built in concrete. A mooring system must be provided for the barge. Storage of the produced oil and other partially processed fluids can be accommodated in the barge. Large floating concrete oil storage facilities have been built in Japan. The size of the barges is influenced by the sea states in which it must operate and the amount of working area it must provide. The water depth in which a barge can operate is a function of the draft of the barge and the operational sea states. A notable production/storage concrete barge is the Ardjuna Sakti liquified natural gas (LNG) barge currently on station in the Java Sea (10). More detailed information on concrete barge-like structures and concrete hulls can be found in (1), (3), and (11).

Other Structures

Concrete subsea oil storage tanks (12,13,14) (Fig. 9) have been proposed for use in waterdepths ranging from 20 to over 400 m (65 to over 1300 ft.). These tanks can be built like the base of a GBS but are fully submerged to the seabed where they function as a gravity base structure.

Concrete wall caissons (Fig. 10) have been used to provide the retaining wall for earth filled islands. These islands provide the working surface for the oil and gas exploration and/or production. The caissons are built as floating units, towed to location, joined into a unit, and then ballasted to the sea floor. The framework of the caissons then forms the perimeter of an island. A hydraulic fill is usually used to fill the interior. When the use of the caisson retained island is complete, the caissons can be refloated, disassembled, and nature allowed to reclaim the island. A notable application was the Tarsuit Caisson Retained Island (15) where the caisson was made of lightweight aggregate concrete. Concrete caissons for an artificial island are a strong contender for development of the Wytch Farm prospect in offshore southern United Kingdom (16). The use of caissons for artificial islands is generally limited to waterdepths of less than 15 m (50 ft.)

Concrete has been used for the base of flare towers and offshore loading buoys. An entire flare boom tower made of concrete will be used for the Sliepner platform in Norway. Concrete anchors (17) for the Snorre TLP have been built. The Maureen offshore development uses a concrete offloading buoy. Concrete subsea wellhead protectors for Oseberg II in the North Sea have been built (18).

The potential for concrete use is great and is limited only by the ingenuity of the concrete designer and constructor.

CONCRETE QUALITY

There is a perception that all concrete used in offshore platforms is something unique and special and requires a technology that is beyond "normal practice" for concrete construction. If "normal practice" means the practice applied to residential construction, the perception is correct. If "normal practice" means the practice applied to any major civil engineering structure such as a building or bridge, then the perception is wrong. There is nothing unique or special in the application of proper batching, delivery, consolidation, and curing of properly proportioned concrete mixtures. In general, the recommended practices for concrete construction, including materials selection and mixture proportioning, that exist in the various building codes, specifications, and standard practices of most developed countries are entirely sufficient for use in the offshore concrete industry. Somewhat different values for water-cementitious ratio, cementing material content, and concrete cover over reinforcing bars may be required because of the marine exposure but these values are well documented. Examples are shown in Tables 2 and 3.

The concrete provided for offshore North Sea platforms has seen a gradual evolution of cube compressive strength from 50 to 70 MPa (7200 to 10100 psi) (19). Table 4 shows the strength development for platforms built by one North Sea contractor. The unique environment in which this concrete is used demands this high quality of concrete. The Ravenspurn North platform (20,21) is in a more moderate environment in the southern part of the North Sea and required only 50 MPa (7250 psi) concrete which was delivered from local ready-mix suppliers. The early concrete platforms made in the Gulf of Mexico used concrete with cylinder compressive strengths from 25 to 35 MPa (3600 to 5000 psi). Recent samples from some 33 year old platforms in the Gulf of Mexico showed an increase in strength from 50 to 69 MPa (5000 to 10000 psi) over the life of the structure (22,23). The actual strength required for a given structure depends on a large number of factors but is significantly influenced by the environmental and operating loads. When these are small, the strength of the concrete can usually be consistent with that which is commonly made in the region of the construction.

CONCRETE MATERIALS

As noted earlier, the constituents of the concrete can be local materials. They must be evaluated, however, to ensure that they have the proper concrete making characteristics and will be durable in the environment in which they are used. Most offshore concrete platforms have a service life of 20 years or more. Because of their offshore location, they are not easily accessible for remedial work when problems occur. To eliminate the high cost of future offshore repair work, the materials used and the resulting concrete must be virtually maintenance free for the service life of the structure.

The durability of offshore and marine Portland-cement concrete is generally defined as its ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration while retaining its original form, quality, and serviceability when exposed to its environment. This includes resistance to deterioration from freezing and thawing action, chemical attack by the constituents of the seawater, physical abrasion due to wave action, floating or suspended solids and debris, and floating ice, corrosion of steel or other metals imbedded in the concrete, and chemical reactions associated with aggregates in the concrete (22). When considering all these deteriorating actions collectively, it is easily deduced that the most aggressive exposure a concrete can routinely experience is in a tidal zone in freezing weather. For most offshore structures, the most prevalent of the destructive mechanisms is the corrosion of reinforcing bars associated with ingress into the concrete of chlorides from the seawater.

Portland cements should have as low of a tricalcium aluminate (C_3A) content as is practical with the local cement production. This helps to reduce the possibility of attacks from sulphates. The total alkali of the cement, calculated as sodium oxide, should not exceed 0.60 percent to minimize any potential for reactivity with the aggregates. The cement should have some finely divided siliceous material added to it (23). This includes natural pozzolans, fly ash, granulated slag, or condensed silica fume. These products contribute to the formation of a denser binder which inhibits the migration of the seawater into the concrete. They also combine with the alkalis to reduce the amount of available alkalis.

Coarse aggregates can be either normal density gravel or crushed stone, or good quality lightweight aggregate. The aggregates should be evaluated with respect to their potential for reactivity with the alkalis in the cement. Those aggregates which are potentially reactive should not be used. Aggregates from areas in close proximity to the sea, should be checked for concentrations of sea salts. These salts must be washed from the aggregate before it is used. Fine aggregates can be either natural or manufactured sands. They, too, must be non-reactive and free from deleterious materials.

In no instance should seawater or brackish water be used to make the concrete. All mixing water should be potable. Washing of aggregates should also be done with potable water.

Chemical admixtures are essential for the production of durable marine concrete. Air entrainment is needed when cycles of freezing and thawing can occur.

High-range water reducing admixtures (HRWRA), commonly called superplasticizers, are required for both consolidation assistance and for improved durability. An HRWRA will allow mixing water reductions up to 30 percent without sacrificing workability. This water reduction significantly reduces the permeability of the concrete and contributes to a densification of the binder fraction of the concrete.

CONCRETE PROPERTIES

Of importance to structural designers are the properties of the materials of construction at an age when appreciable loads are applied to the structure. For most offshore structures, the maximum loadings occur when the structure is put into service. This can vary from 1 to 5 years from the start of construction depending on the size and complexity of the structure and its ultimate use. The properties of hardened concrete that are used by the designers of offshore concrete platforms are:

a. compressive strength,
b. tensile strength,
c. modulus of rupture,
d. modulus of elasticity,
e. Poisson's Ratio,
f. stress-strain relationships,
g. fatigue strength,
h. absorption,
i. shear strength,
j. creep and shrinkage,
k. shear friction capacity,
l. bearing strength,
m. and, thermal properties such as the coefficient of thermal expansion, thermal conductivity, specific heat, and diffusivity.

The numerical value of each of these properties is generally not critical because the design process can usually use whatever values the selected concrete produces. The specific properties may not always be complimentary, however. For example, a very high compressive strength concrete (e.g., 65 MPa (9400 psi)) may allow compressive structural members to be reduced in cross-section for a given loading. If, however, the corresponding increase in the modulus of elasticity of that concrete allows cracking to occur at lower strain strain levels, then additional reinforcement may be required to reduce the cracking. Because the cross-section has now been reduced, the additional reinforcing steel adds to the congestion within the wall and makes the concrete placement more difficult. The cost of the in-place reinforcing steel may also be more than the reduction in cost due to using less concrete. Trade-off's between the various properties of the concrete should be attempted, where possible, to achieve the most efficient and cost-effective design.

All of the hardened concrete properties should be determined at advanced ages for the specific concrete to be used in an offshore structure. Unfortunately, this is not always possible and early age properties (e.g., at 28-days age) are often used. This gives the design a conservative flavor but it may add substantial costs to the structure. There is a risk associated with extrapolating early age data, particularly with high strength concretes, because the improvement of concrete properties with age may not always follow assumed trends.

Other properties of the concrete are of concern to the constructor rather than the designer. These include:

a. workability,
b. pumpability,
c. unit weight,
d. air content,
e. consolidation,
f. thermal gradients,
g. and, finishing.

The inter-relationship of these properties is a complex problem. Of utmost importance is the unit weight of the concrete. For a structure of given dimensions and configuration, and that may also be required to carry a fixed amount of dead load while floating, variations in the concrete unit weight may adversely affect the floating stability of the structure, causing it to sink or overturn. The in-place unit weight, in turn, is affected by the mixture ingredients, their proportions, and the void content, which is both a function of the entrained air content and the entrapped air or voids remaining after consolidation. If the mixture does not have adequate workability to surround the high levels of reinforcing bars that may occur, additional voids could result in the concrete. The absorption values determined on the hardened concrete are applied to the hardened density of the concrete to establish what the concrete density is when the structure is in the water. If the actual density varies significantly, so will the actual absorption values which will be different than those used in the design process.

As described later, the typical structural members in an offshore platform are quite thick. Because most offshore codes require fairly high cement contents (see Table 2) for durability purposes, the possibility of significant heat development within the concrete exists. Limiting values for the maximum placing temperature and the maximum heat rise are contained in the Codes. Even when meeting these requirements, care must be exercised to minimize thermal gradients so that thermal cracking of the structural members does not occur.

The finish of the concrete surface of an offshore structure may seem like a non-critical item, but a poor finish can have several undesirable effects beside appearance. For most offshore structures, the governing design load is caused by the forces from sea waves acting on the surface of the structure. Rough surfaces tend to gather more wave forces and thus reduce the factor of safety planned for a structure. In cold climates, an initially rough surface tends to degrade faster when subjected to cycles of freezing and thawing because there are receptacles in the surface of the concrete for water to collect and freeze. In ice-infested waters, ice moving against and past a structure tends to abrade rough surfaces faster than smooth surfaces (23).

Other properties of the concrete that are usually not of concern to either the designer or the constructor, are the durability properties. These are of concern to the owner as the offshore structure is usually part of a profit making venture that has a prescribed life-time. Some of the durability aspects of the concrete, such as freezing and thawing resistance, are addressed in the Code requirements. Matters such as the air void system in hardened concrete, as defined by spacing factor, specific surface, and voids per millimeter (inch), have specific requirements which must be met. Guidance is also provided in the Codes to prevent or mitigate such deleterious effects as sulfate attack and alkali-aggregate reactivity through proper materials selection.

Chloride-ion permeability of the concrete should also be evaluated to insure that a satisfactory concrete is being provided to resist reinforcing bar corrosion. Although minimum concrete cover over the reinforcing bars is specified for a given exposure zone (see Table 3), this may have to be increased if the concrete to be used in the platform does not have adequate resistance to chloride-ion penetration.

The abrasion resistance of the concrete to water-borne sediments, debris, floating objects, and ice is usually not specified as it is a rather site-specific phenomena. In offshore areas where significant abrasion can occur, such as ice-infested waters, the resistance of the concrete to the abrading medium must be evaluated and loss rates for the concrete surface determined (24). Once these rates are known, measures to accommodate or eliminate the losses, such as additional concrete cover or steel plates in the abrasion zone, respectively, can be implemented.

An evaluation of all the concrete properties noted above for a specific concrete for a specific structure is the ideal situation, but it has not often been done. When actual numerical values are not available, conservative approximations are chosen and these result in a satisfactory, but not necessarily cost efficient, design. One study that addressed most of the properties noted above, was performed on high strength lightweight aggregate for use in offshore Arctic structures and is described in (25) to (27).

DESIGN CONSIDERATIONS

As noted above, concrete offshore structures can be bottom-founded or floating. With the exception of a structure which has its base made entirely from prestressed concrete piles, most of the other bottom-founded structures are in a floating mode at some time in their early life. These structures must then include design provisions for both bottom-founded operational loads as well as those loads associated with the structures behavior as a ship.

Design codes and guidelines for offshore concrete structures have been developed by various regulatory agencies and standards groups. A listing of some of the major codes and regulations is shown in Table 5. These are constantly being upgraded as the technology advances. In general, detail design of the individual elements of an offshore concrete structure for such things as shear, tension, flexure, compression, eccentric loads, etc., is not significantly different than for any other type of concrete structure. It is only the types of loads, their frequency and duration, and their magnitude that differs from ordinary civil engineering structures.

The principal loads the offshore structure encounters are permanent loads, variable functional loads, environmental loads, accidental loads, and deformation loads. These various loads are combined in realistic manners to determine their net effect.

Permanent loads include the weight of the structure, any permanent equipment, ballast that will not be removed, and the external hydrostatic sea water up to mean sea level. Variable functional loads are the loads associated with the normal operation of the structure. Loads in this category that are unique to offshore structures include variable ballast, installation and drilling loads, vessel impact, fendering and mooring, weight of petroleum products temporarily stored in the platform, helicopter loads, and crane operations.

Environmental loads include waves, wind, current, ice and snow, and earthquake. Accidental loads include fire, explosion, ship impact, unintentional flooding, unintentional ballast distribution, and changes in presupposed pressure differences. Examples of deformation loads include prestressing, concrete shrinkage, and thermal gradients.

The geotechnical considerations offshore are much more complex than onshore. For bottom-founded structures, this is an extremely critical area of design. The anchors and moorings of floating structures are also significantly influenced by the subsea soil conditions. Seismically active areas warrant special consideration. Specialists in subsea foundation problems, not onshore foundation specialists, should always be used to work this part of the design problem.

For the initial offshore concrete structure in a country or region that has never used one before, it is desirable to use the design expertise of companies or firms that have prior experience with these structures. Such firms exist in North America, Europe, Scandinavia, United Kingdom, and Japan. By involving local design firms in partnerships with these experienced firms, the philosophy and mechanics of the design process can be transferred to the local regions.

SAFETY CONSIDERATIONS

Modern offshore concrete platforms are designed with sufficient redundancy to resist major accidental loads. Concrete has exceptionally good impact resistance and only a few isolated instances of structural damage due to ship impact have been reported. Sufficient ductility can be designed into structural concrete elements to eliminate the problem of progressive collapse. The fire resistance of concrete is well known with concrete often being used to protect steel from fires in many major structures. A summary of the service record of concrete platforms in the North Sea can be found in (28).

Floating concrete structures are designed for one compartment damage stability which means that any local damage which causes leakage of the sea into the hull will not cause the floating structure to be at risk. Similar criteria are applied to the bottom-founded structures so that they are not at risk while in their floating mode. Bottom-founded structures, such as that shown in Fig. 1b, have the unique capability of having each of its shafts or towers operate independently of the others. All living quarters and other major personnel areas can be isolated on top of one shaft and kept removed from the more dangerous areas where drilling and processing of hydrocarbons takes place. All areas can be connected by bridges. This is a significant advantage over a structure where most of the supporting structural members are tied together in some fashion and collectively support all the operations of the platform. In the event of a major fire or explosion in or on the shafts where the hydrocarbons are present, the living quarters would not be affected and successful evacuation of the GBS type platform could take place.

CONSTRUCTION PRACTICES

There are no unique construction practices needed to build an offshore structure such as those described above. The good practices employed for any major civil engineering project are sufficient for building an offshore concrete platform. This gives the owner flexibility in selecting construction contractors.

The large offshore concrete platforms of the North Sea have been predominately constructed using slipforming (29). It has been demonstrated that the North Sea slipforming techniques can be satisfactorily applied to offshore concrete construction in the hotter climate of the Gulf of Mexico (30). The small barge-like platforms in the Gulf of Mexico are cast-in-place (3). The Hibernia platform (31) for the East Coast of Canada, which is scheduled for completion in 1995, will be jump-formed. Precast elements have been used in some structures (32). The method of construction can be anything that works and should be left to the discretion of the contractor.

The concrete can be delivered to the form by pump, boom, conveyor, bucket, buggy, barrow, or, again, anything that works. Sophisticated distribution systems normally are not needed. The distribution of the concrete into the slipforms used on North Sea structures has been by wheel-barrow.

Proper and sufficient consolidation of the concrete is essential. The equipment and procedures to do this already exist. Adequate curing must be provided. This includes protection from early freezing in regions where this is a possibility.

The two items of offshore concrete construction that differ from most onshore concrete construction, are the thickness of the concrete elements and the amount of reinforcing steel that is used. The thinnest concrete walls are usually 350 mm (13 in.). The thickest walls can be several meters (greater than 6 feet) thick. Typical wall thickness's are 500-600 mm (20-24 in.).

Temperature control of the concrete in these thick walls is essential to eliminate thermal cracking problems. Reinforcement densities typically average 400 kg/cu m (676 lb/cu yd). Extremes in critical areas have been as great as 1100 kg/cu m (1859 lb/cu yd). The proportioning of the concrete and the consolidation methods must be tailored to insure the reinforcing steel is completely encapsulated by the concrete.

In general, the maximum size aggregates used in these structures has been 19 mm (3/4 in.) or less to allow the concrete to move around the large amounts of reinforcing bars. The use of high-range water reducers (superplasticizers) is a necessity.

Prestressing is also used in almost every structure. This is to insure water tightness of the concrete. The amounts of prestressing are structure and location specific. Standard prestressing materials and practices can be used for these structures with a view that the work will be done in a marine environment. Special care must be taken to protect end anchorages of the prestressing from the corrosive environment of the sea.

CONSTRUCTION LOCATIONS

Bottom-founded and floating offshore concrete structures can be constructed, either partially or completely,

1. In dry docks or graving docks,
2. On submersible barges,
3. On skid ways, or
4. In precast facilities onshore.

Precast facilities are used principally for the precast concrete piles and template decks. These facilities can also be used to prefabricate substantial portions of a structure. The prefabricated elements are then transported to and assembled at the other locations. The construction of the concrete Arctic drilling structure Glomar Beaufort Sea I, also known as the CIDS (Concrete Island Drilling System), had most of its interior concrete elements made in a prefabrication facility (32). The actual method selected will depend greatly on the existing site facilities where construction is planned and on the economics of the project.

Dry Dock Construction

The principal method of construction used for the large North Sea concrete platforms is to begin the construction in a dry dock or graving dock. Fig. 11 shows the construction sequence. Once the base of the structure becomes stiff enough and has sufficient buoyancy, the dock is flooded and the base allowed to float. It is then towed from the graving dock to a deeper water location where it is temporarily moored. Construction then continues at that location. Depending on available water depths, this location can be close to the shore or a great distance away. If the structure is located close enough to shore to be reached by a floating or fixed bridge, materials and personnel can reach the structure directly from the shore. Office facilities and concrete production facilities can remain onshore. If the structure is located some distance away from the shore, it will require adjoining barges which support offices, materials laydown areas, concrete batching plants, and other essential equipment. Concrete materials are obtained directly from supporting ships. Work crews are shuttled back and forth from the shore by crew ships.

Once the concrete construction is complete, the structure may be moved to an even deeper water location where it is mated with the topsides equipment and hook-up of the equipment is begun.

It generally costs less to construct a GBS entirely in a dry dock rather than partially in the dry dock, at a wet dock or at a jetty mooring. Productivity is generally higher in the dry dock and specialized equipment is minimized. Construction risk is usually lower. For smaller concrete platforms, the platform can be built entirely in the dry dock with sufficient buoyancy to be floated out and towed to its final location. The Ravenspurn North platform (19,20) and all the barge-like platforms (Fig. 3 and 4) built for the Gulf of Mexico are constructed in that manner. Fig. 12 shows an exploded view of various components of the Ravenspurn North platform (33). It is reported (34) that in Europe, concrete GBS's for water depths from 100 to 150 m (330 to 490 ft.) can be constructed entirely in a dry dock. The limit is governed by a combination of gate width and sill draft. As noted above, the total support for complete dry dock construction is land-based and very similar to constructing a concrete building. Very large platforms can also be built in this manner but will require the use of auxiliary buoyancy compartments for floatation. These compartments can be designed to be removed after platform installation or can be left in place. Some small platforms have also used additional buoyancy compartments to satisfy installation requirements.

A major expense in the construction of a concrete platform is the development of a dry dock if one does not exist, or the modification of existing dry dock facilities to accommodate the construction. This expense can include land procurement, excavation, cofferdam construction, dewatering systems, dredging of channels for float out, construction of supporting quays, docks and wharves, and the overall upgrading of the infrastructure to improve project support (roads, bridges, power supply, water supply, sewage treatment, etc.). These costs can easily reach 80 percent of the project cost in remote areas.

Construction on Barges

To eliminate much of the costs associated with the construction of a dry dock, the construction of some or all of the platform on submersible barges is a practical solution. This eliminates the need for a dry dock and thus greatly expands the potential for construction site locations. Fig. 13 shows a typical barge construction scenario. Any location with sufficient water depth for barge and supply ship operation is a possible construction site. Preferably this site should be in sheltered waters and not subject to severe seas. The number of barges required will depend on the size of the structure and the capacity of the barges. Special built barges are always a solution but it may be more economical to weld together a sufficient number of smaller, standard size barges to accomplish the same objective. Once the barge or barges have been assembled to provide the working platform, construction then proceeds on the barge(s) as if it was on land. The construction support will depend on the water depths available. Initially, the construction can probably begin with the barges moored adjacent to the shore so that all construction support can come directly from the land. Fig. 14 shows the concrete tether anchors for the Snorre Tension Leg Platform (TLP) which were constructed entirely on barges with direct land support. If the platform is small, the entire platform can be built on the barges at one location. If the platform is large, only a portion would be built on the barges. That portion would be floated off the barges, temporarily moored, and construction completed while in a floating mode.

Skidway Construction

Skidways exist in almost all marine fabrication yards. A skidway is basically a structural slab having a small slope that extends from a construction area down into the sea. Structures that are built on the skidway can be gravity assisted as they are moved down the slope and into the sea. The structures can be self-floating or can be skidded onto barges.

For a concrete GBS, the structure can be built either in part or in its entirety on the skidway (35). Small structures (Fig. 15) are the most likely candidate for this type of construction. Because the weight of a complete large concrete structure is so large, there is probably an upper limit where any given skidway could not be used without significant structural upgrading. This upgrading may not be cost effective but warrants consideration. The base of most concrete structures can probably be constructed on the skidway to a level where it would be self-floating. It can then be skidded into a floating mode and towed to a deeper water site for mooring and completion while floating.

Site Limitations

The principal limitation for any of the construction sites, is its access to the open sea or to maintained ship channels. As most of the structures will float at some time during their construction and installation phases, water of sufficient depth for them to float and to be towed to their final location must exist in close proximity to the construction site. Dredging of new channels is very expensive and may not be possible due to physical, environmental or political reasons. Most large structures with deep drafts have been completed while in a floating mode. When this is done, the deep water requirement is further constrained because of a need to have this deep water location in a relatively mild environment with respect to wind, waves and current. It is also desirable to have this deep water location close enough to shore so that materials supply and construction crew changes can be made without having to stockpile large amounts of material offshore or to provide temporary offshore quarters for personnel. Both of these aspects of construction will add greatly to the cost of the project.

If a dry dock is required and must be built, the supporting infrastructure must be carefully studied before a site is selected. If the site is not convenient to major population areas, the site development will have to include such things as accommodations for work crews, dining facilities, water supply, sewage disposal, recreation facilities, communication facilities, transportation and all the other things required for a small city.

As noted above, the existence of skidways in operating shipyards does not necessarily mean they can be used for concrete structures because they may be undersized or structurally inadequate. The dredged channels leading away from skidways most likely will not be of sufficient depth for the draft of larger concrete platforms and some additional dredging may be required.

MARINE OPERATIONS

The subject of marine operations is outside the scope of this paper, but it should be noted that offshore development and marine operations go hand-in-hand. The construction and installation of any type of offshore concrete platform will involve significant marine operations (36). The structure itself must float, be towed, and perhaps submerged for final installation.

The structure may have its topsides equipment installed in a mating operation. In this operation, the structure is ballasted down until it is almost submerged. The assembled topsides is then floated over the submerged structure. Deballasting is initiated, and as the structure rises, it picks the topsides up off the barges. Deballasting is continued until the structure reaches its final draft for towing operations. Hook-up of the topsides to the structure is then done. This mating is a critical marine operation.

Marine operations are expensive and may require special vessels. Fortunately, many good specialist firms or consultants with offshore structure installation experience exist around the world and they should be consulted on this aspect of the use of concrete platforms.

COST CONSIDERATIONS

The costs of an offshore structure are usually dictated by the complexity of the structure, the location where it will be built, and the methods of construction used. These structures have the potential for low capital costs because the materials are local and the labor skill level usually does not have to be high. When properly made, concrete can be virtually maintenance free thus producing an attractive life-cycle cost. Detailed cost discussions are beyond the scope of this paper but a cost philosphy for these structures is described in (37) and (38).

SUMMARY

The above information is intended to provide a general overview of what is required in the use of concrete for offshore structures. A significant amount of information on the subject is distributed throughout the literature but has not yet been assembled in a useful text book form.

Concrete is the material of choice for permanent constructions in all regions of the world. It can be produced with local labor and local materials. The potential exists for use in the offshore oil and gas industry in developing regions of the world and in existing offshore regions where steel structures have been predominately used before. The use of concrete for these structures generally must be sold to owners or developers who are used to dealing with steel structures.

For use in offshore structures, the concrete technology is well established. No special materials are required, the concrete quality is not unusual, design requirements are well established, competent offshore concrete design firms exist in many regions of the world, no special construction practices are needed, flexibility exists in construction site selection, capital costs can be low, and life-cycle costs can be very attractive. Because of the severe environment that exists offshore, the durability aspects of the concrete will require special attention.

The potential for the use of concrete in offshore structures is great and is limited only by the ingenuity of the concrete designer and constructor and their ability to sell their ideas to the owners.

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TABLE 1--SUMMARY OF FLOATABLE/BOTTOM-FOUNDED CONCRETE HULL STRUCTURES FOR THE GULF OF MEXICO CONSTRUCTED BY ONE CONTRACTOR BETWEEN 1984 AND 1989 (4)

Year Installed	Location	Water Depth m (ft.)	Hull Dimensions width x length x height (m x m x m)	Hull Dimensions width x length x height (ft x ft x ft)
1984	Eugene Island Block 45	6.1 (20)	17.7x17.7x3.7	(58x58x12)
	Bayou Sorrel	2.4 (8)	9.4x11.0x3.7	(31x36x12)
	Cox Bay North	2.1 (7)	19.5x46.9x3.0	(64x154x10)
	Cox Bay South	2.1 (7)	19.5x33.5x3.0	(64x110x10)
	Vermilion Block 72	7.0 (23)	17.7x17.7x3.7	(58x58x12)
	Eugene Island Block 45	7.0 (23)	18.3x18.3x3.7	(60x60x12)
1985	Brenton Sound[1] Block 1	2.7 (9)	18.3x26.8x3.7	(60x88x12)
	Lease Platform	NK	8.5x30.5x3.7	(28x100x12)
	Eugene Island Block 44	6.4 (21)	17.7x17.7x3.7	(58x58x12)
	South Pass	2.4 (8)	21.9x49.4x3.7	(72x162x12)
1986	Gordon Island Bay	NA	10.7x19.8x3.7	(35x65x12)
	Lease Platform	NK	8.5x30.5x3.7	(28x100x12)
	Brenton Sound[2] Block 2	3.7 (12)	18.3x24.4x3.7	(60x80x12)
	Quarantine Bay	NK	21.3x38.7x3.7	(70x127x12)
1987	Delta Dock Fld	NA	12.2x30.5x3.7	(40x100x12)
	Pt. Ala Hache	NK	21.3x40.8x3.4	(70x134x11)
	Pt. Ala Hache	NK	28.0x32.9x3.4	(59x108x11)
	W. Lake Verret	NK	15.2x29.9x3.7	(50x98x12)
1988	Chandeleur Snd.	NA	4.6x7.6x3.0	(15x25x10)
	Atchsduya Bay[3]	NA	15.5x15.5x4.3	(51x51x14)
	S. Marsh Island Block 253	NK	18.3x18.3x3.4	(60x60x11)
1989	Main Pass Block 69	NA	12.2x18.3x3.7	(40x60x12)
	Eugene Island Block 30	4.3 (14)	21.9x25.0x4.0	(72x82x13)
	West Bay	NK	21.9x42.7x3.7	(72x140x12)

NK = not known.
NA = not applicable. Barge structure.

[1] Designed for 5000 bbl of storage.

[2] Designed for 4500 bbl of storage.

[3] Designed for 2500 bbl of storage.

TABLE 2--SUMMARY OF RELEVANT CODE REQUIREMENTS FOR WATER/CEMENT RATIO, CEMENT CONTENT, AND COMPRESSIVE STRENGTH

	Exposure Zones		
Codes (See Table 3)	Submerged	Splash	Atmospheric
ACI 357			
Maximum water/cement ratio	0.45	0.40	0.40
Minimum cement content, kg/cu m (lb/cu yd)	356 (600)	356 (600)	356 (600)
Maximum cement content, kg/cu m (lb/cu yd)	415 (700)	415 (700)	415 (700)
Minimum 28-day cylinder compressive strength, MPa (psi)	35 (5000)	35 (5000) 42(6100) if subject to abrasion	35 (5000)
CSA S474			
Maximum water/cement ratio	0.45	0.40	0.40
Minimum quantity of cementing material, kg/cu m (lb/cu yd)	360 (610)	400 (675)	400 (675)
Minimum quantity of portland cement, kg/cu m (lb/cu yd)	300 (510)	300 (510)	300 (510)
Minimum cylinder compressive strength, MPa (psi),			
28 days	30 (4350)	40 (5800)	40 (5800)
91 days	35 (5000)	45 (6500)	45 (6500)
FIP			
Maximum water/cement ratio	0.45 (less than or equal to 0.40 preferred)		
Minimum cement content, kg/cu m (lb/cu yd)	320 to 360* (540 to 610)	400 (675)	320 to 360* (540 to 610)
Maximum cement content, kg/cu m (lb/cu yd)	500 (845)	500 (845)	500 (845)
Minimum 28-day cylinder compressive strength, MPa (psi)	32 (4650)	32 (4650) 40(5800) if subject to abrasion	32 (4650)

(continued)

TABLE 2--CONTINUED

Codes (See Table 3)	Exposure Zones Submerged	 Splash	 Atmospheric
DnV			
Maximum water/cement ratio	0.45 (less than or equal to 0.40 preferred)		
Minimum cement content, kg/cu m (lb/cu yd)	300 (510)	400 (675)	300 (510)
Maximum cement content, kg/cu m (lb/cu yd)	---	---	---
Minimum 28-day cylinder compressive strength, MPa (psi)	---	---	---
BS 6235			
Maximum water/cement ratio	0.40	0.40	0.40
Minimum cement content, kg/cu m (lb/cu yd)	320 to 360* (540 to 610)	400 (675)	400 (675)
Maximum cement content, kg/cu m (lb/cu yd)	---	---	---
Minimum 28-day cylinder compressive strength, MPa (psi)**	32 (4650)	32 (4650) 40(5800) if subject to abrasion	32 (4650)

* 320 kg/cu m (540 lb/cu yd) for a maximum aggregate size of 40mm (1.57 in.); 360 kg/cu m (610 lb/cu yd) for a maximum aggregate size of 20mm (0.78 in.).

** Cylinder strength assumed to be 80 percent of specified cube strength.

TABLE 3--SUMMARY OF RELEVANT CODE REQUIREMENTS FOR MINIMUM CONCRETE COVER

Codes (See Table 5)	Splash or External Atmospheric	Other
ACI 357		
Untreated reinforcing bars	65 mm (2.6 in.)	50 mm (2.0 in.)
Prestressing tendons	90 mm (3.5 in.)	75 mm (3.0 in.)
Cover of stirrups	13 mm (0.5 in.)	less than above
CSA S474		
Untreated reinforcing bars	65 mm (2.6 in.)	50 mm (2.0 in.)
Epoxy coated reinf. bars	50 mm (2.0 in.)	35 mm (1.4 in.)
Prestressing tendons	90 mm (3.5 in.)	75 mm (3.0 in.)
	A cover of 75 mm (3.0 in.) may be used in atmospheric zone.	
Cover of stirrups	15 mm (0.6 in.)	less than above
FIP		
Untreated reinforcing bars	65 mm (2.6 in.)	50 mm (2.0 in.)
Prestressing tendons	90 mm (3.5 in.)	75 mm (3.0 in.)
DnV		
Untreated reinforcing bars	50 mm (2.0 in.)	40 mm (1.6 in.)
Prestressing tendons	100 mm (4.0 in.)	80 mm (3.2 in.)
BS 6235		
Untreated reinforcing bars	75 mm (3.0 in.)	60 mm (2.4 in.)
Prestressing tendons	100 mm (4.0 in.)	75 mm (3.0 in.)

TABLE 4--STRENGTH DEVELOPMENTS FOR NORTH SEA OFFSHORE CONCRETE STRUCTURES (18)

Platform (Year)	Concrete in Cell Walls* m^3 (yd^3)	Specified Concrete Grade	Obtained 28-Day Cube Strength MPa (psi)	Typical Slump mm (in.)
Ekofisk I (1972)	---	C40*	45* (6530)	100 (3.9)
Beryl A (1974)	17100 (22370)	C45	55.0 (7980)	120 (4.7)
Brent B (1974)	40600 (53100)	C45	53.0 (7690)	120 (4.7)
Brent D (1975)	34000 (44470)	C50	54.2 (7860)	120 (4.7)
Statfjord A (1975)	47400 (62000)	C50	54.6 (7920)	120 (4.7)
Statfjord B (1979)	56700 (74160)	C55	62.5 (9070)	160 (6.3)
Statfjord C (1982)	63700 (83320)	C55	67.5 (9790)	210 (8.3)
Gullfaks A (1984)	63400 (82930)	C55	65.2 (9460)	220 (8.7)
Gullfaks B (1985)	45000 (58860)	C55	80.8 (11720)	220 (8.7)
Oseberg A (1986)	43000 (56240)	C60	76.7 (11120)	230 (9.1)
Gullfaks C skirts (1986)	17400 (22760)	C70	83.8 (12150)	240 (9.4)
Gullfaks C (1989)	115000 (150420)	C65	79.0 (11460)	230 (9.1)

* Only the slipformed concrete in the cell walls except where noted. Does not represent the total concrete in the structure.

TABLE 5--SUMMARY OF RELEVANT CODES FOR OFFSHORE CONCRETE STRUCTURES

1. American Concrete Institute ACI 357R-84 (revised 1989), Guide for the Design and Construction of Fixed Offshore Concrete Structures, Detroit, Michigan, USA.

2. Canadian Standards Association (CSA), Code for the Design, Construction, and Installation of Fixed Offshore Structures, Part 4, Preliminary Standard S474-M1989, Concrete Structures, and Special Publication S474.1-M1989, Commentary to S474-M1989, Rexdale, Ontario, Canada.

3. Federation Internationale de la Precontrainte (FIP), Design and Construction of Concrete Sea Structures, 4th Edition, 1985, The Institution of Structural Engineers, London, England.

4. Det norske Veritas (DnV), Rules for the Design, Construction, and Inspection of Offshore Structures (with appendices), 1977, Hovik, Norway.

5. British Standards Institution BS 6235, Code of Practice for Fixed Offshore Structures, 1982, London, England.

6. Det norske Veritas Classification A/S, Rules for the Classification of Fixed Offshore Installations Structures, 1989 (draft), Hovik, Norway.

7. American Bureau of Shipping (ABS), Rules for Building and Classing Offshore Installations, 1983, New York, New York, USA.

8. American Petroleum Institute (API) RP 2A, Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms, Eighteenth Edition, 1989, Washington, DC, USA.

9. American Petroleum Institute (API) RP 2N, Recommended Practice for Planning, Designing and Constructing Fixed Offshore Structures in Ice Environments, Second Edition, 1988, Washington, DC, USA.

10. Federation Internationale de la Precontrainte (FIP), Design and Construction of Concrete Concrete Ships, 1986, The Institution of Structural Engineers, London, England.

11. Norwegian Petroleum Directorate (NPD), Acts Regulaons and Provisions for the Petroleum Activity, 1985, Oslo, Norway.

12. Norwegian Petroleum Directorate (NPD), Regulations Concerning Load Bearing Structures in the Petroleum Activity (Draft), 1990, Oslo, Norway.

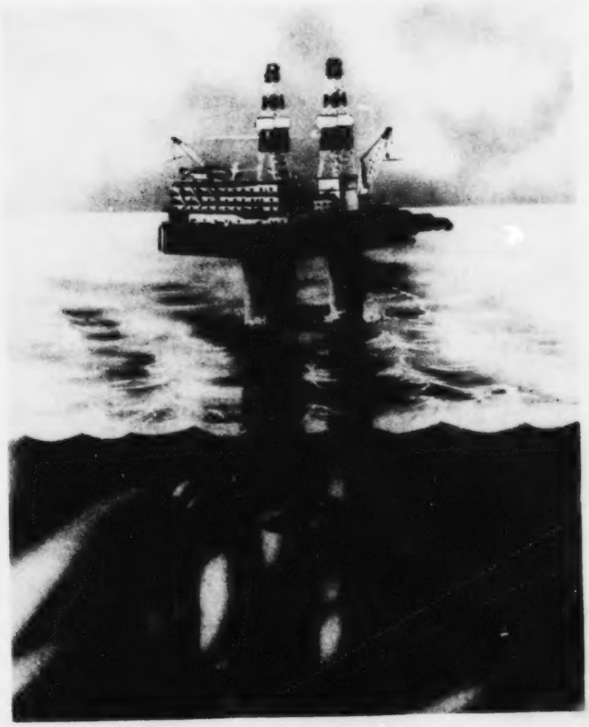


Fig. 1. Typical Gravity Base Structure

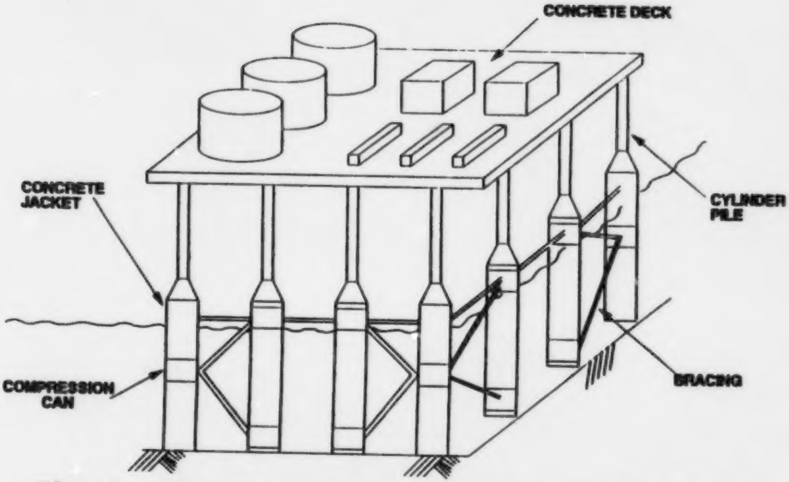

CONCRETE DECK
CONCRETE JACKET
CYLINDER PILE
COMPRESSION CAN
BRACING

Fig. 2. Typical Cylinder Pile Supported Platform

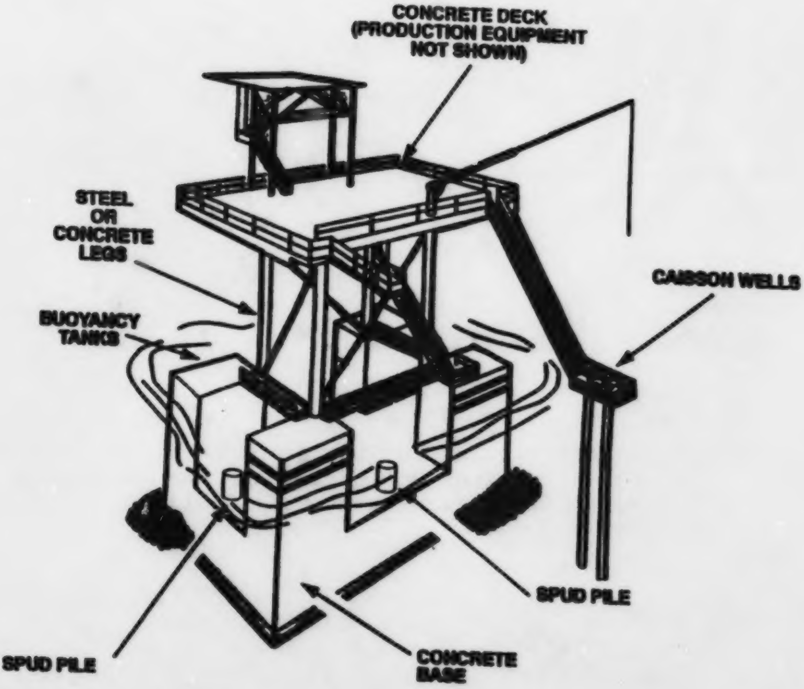

CONCRETE DECK (PRODUCTION EQUIPMENT NOT SHOWN)
STEEL OR CONCRETE LEGS
CAISSON WELLS
BUOYANCY TANKS
SPUD PILE
SPUD PILE
CONCRETE BASE

Fig 3. Floatable/Bottom-Founded Structure (Courtesy of Production Management Structural Systems)

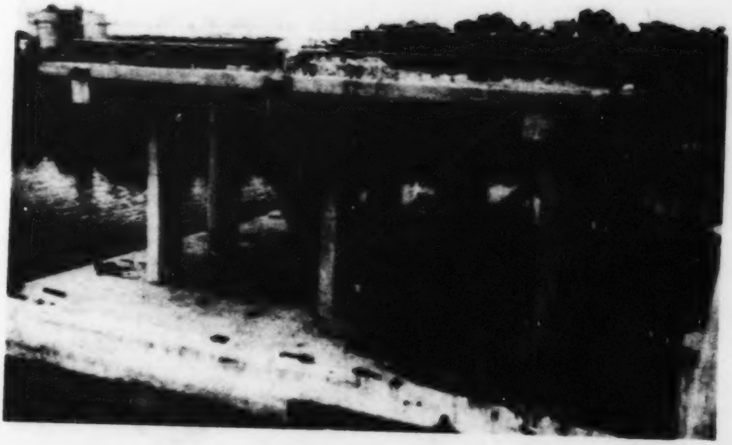


Fig. 4 Eighteen-year Old Floatable/Bottom Founded Platform After Refloating and Relocation to Wet-Dock for Equipment Modifications

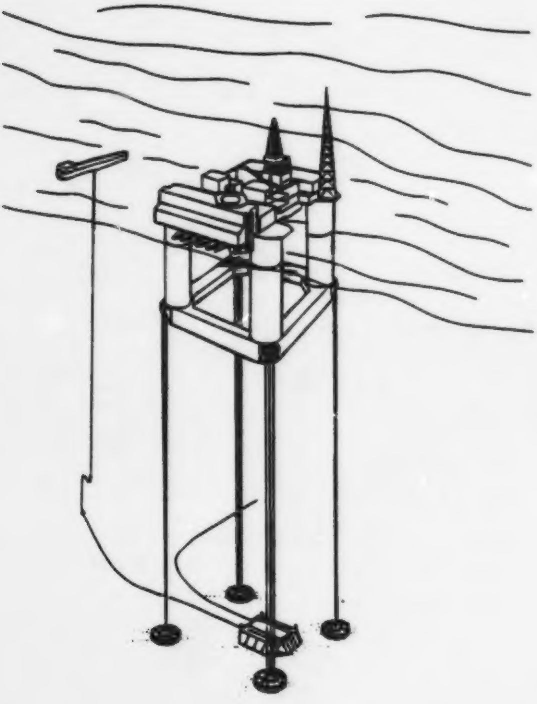


Fig. 5. Concrete Tension Leg Platform (TLP)

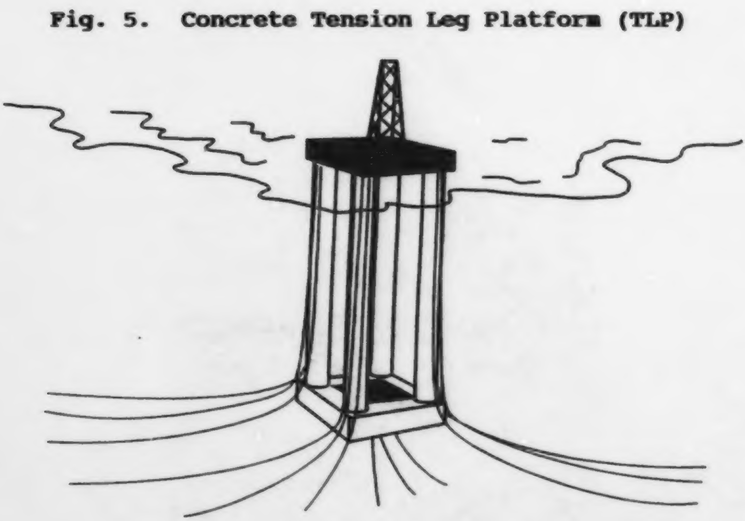


Fig. 6. Deep Draft Concrete Floater (DDCF)

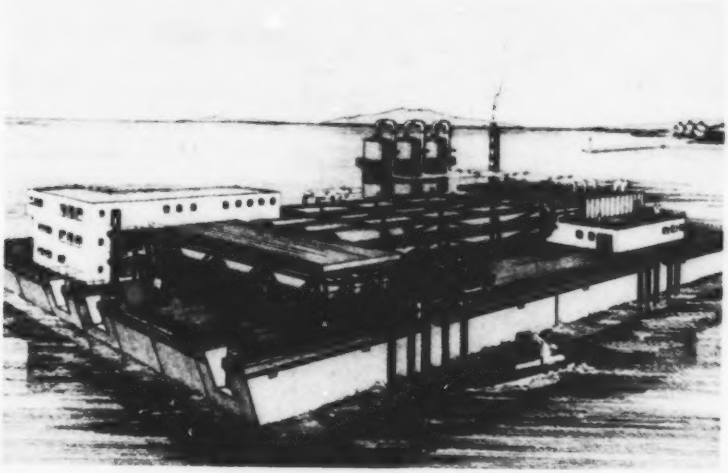


Fig. 7. Concrete Production Barge (Courtesy of Ed. Zublin AG)

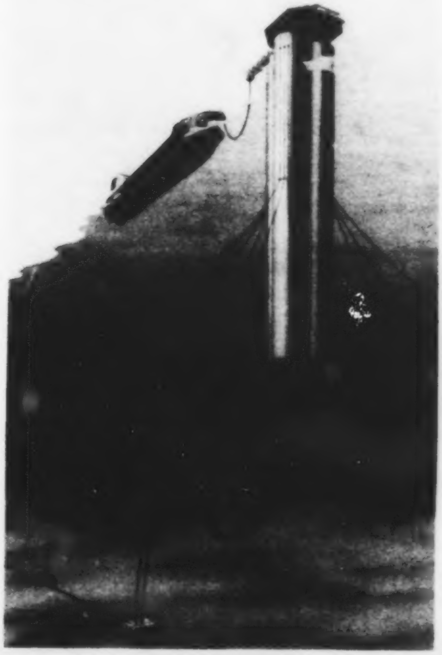


Fig. 8 Spar Buoy Platform (Courtesy of Norwegian Contractors)

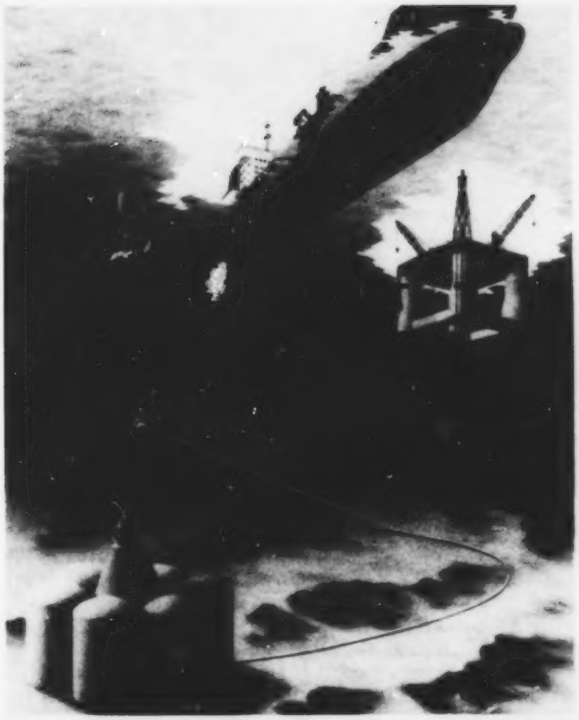


Fig. 9. Concrete Subsea Storage Tank

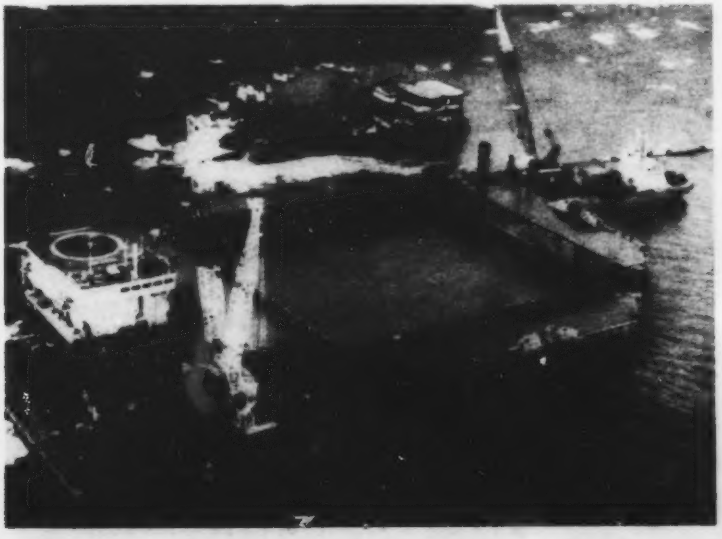
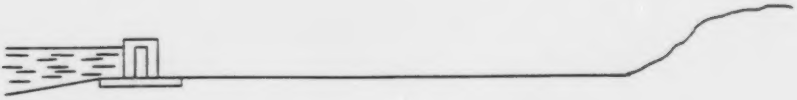
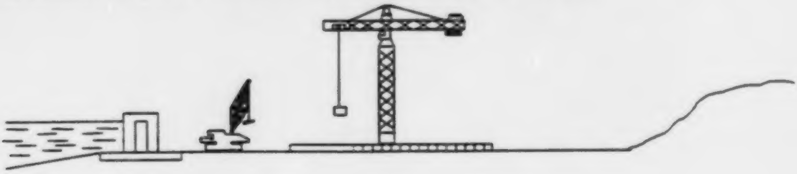


Fig. 10. Concrete Retaining Wall Caissons (14)

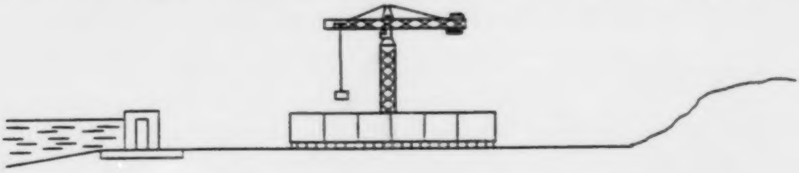
1. EXCAVATION OF CONSTRUCTION AREA



2. CONSTRUCTION OF SUBBASE OR BASE SLAB OF PLATFORM



3. CONSTRUCTION OF CONCRETE PLATFORM TO SUFFICIENT HEIGHT FOR TOW-OUT



4. FLOODING OF DRY DOCK

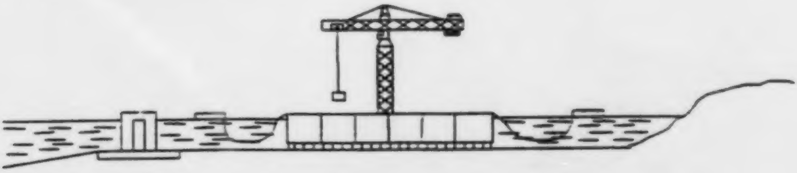
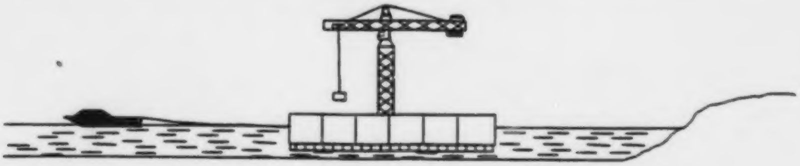
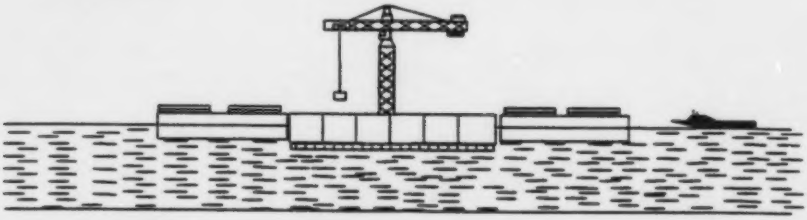


Fig. 11. Dry Dock Construction Scenario (continued)

5. TOW-OUT OF PLATFORM TO DEEPER WATER CONSTRUCTION SITE



6. CONTINUATION OF CONSTRUCTION OF FLOATING BASE FROM FLOATING WORK BARGES



7. CONTINUED CONSTRUCTION UNTIL BASE IS COMPLETE

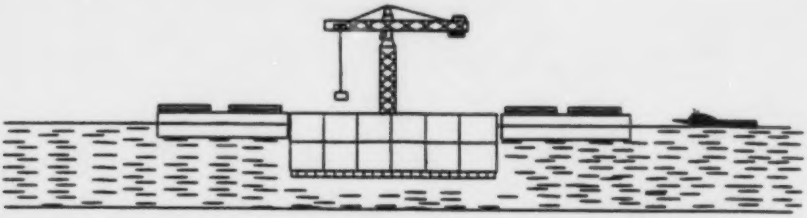


Fig. 11 (cont.). Dry Dock Construction Scenario

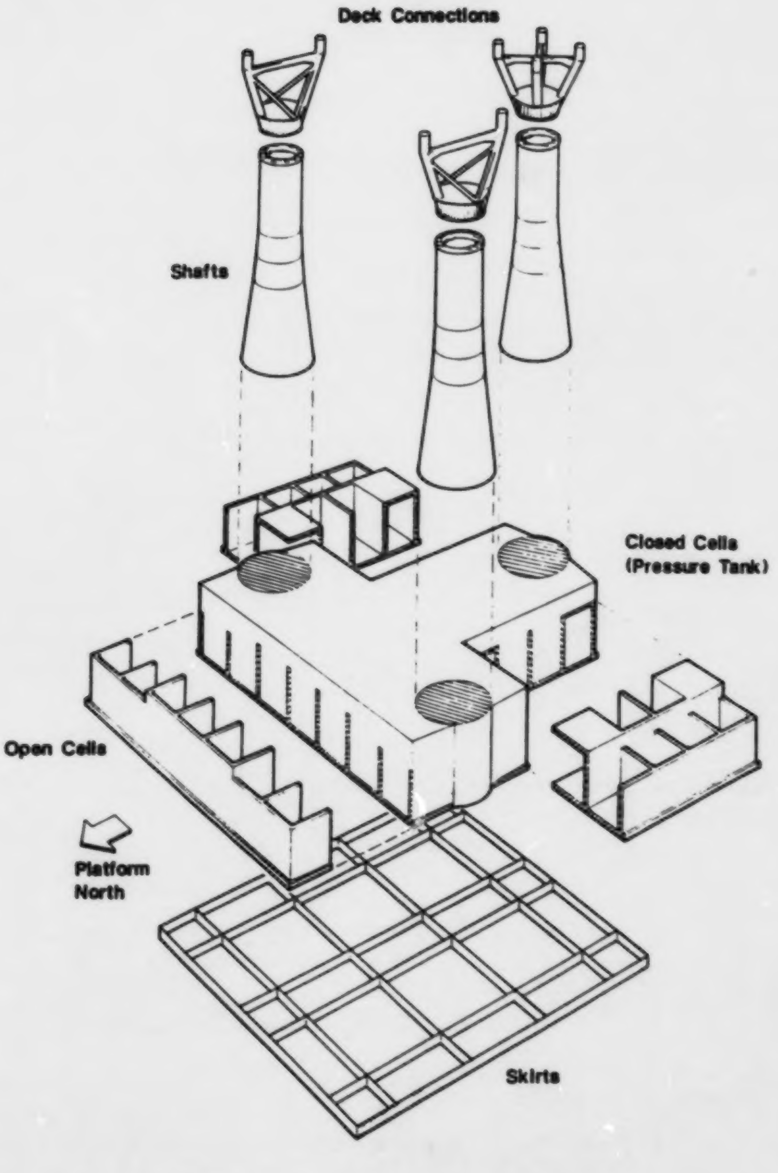

Deck Connections
Shafts
Closed Cells
(Pressure Tank)
Open Cells
Platform
North
Skirts

Fig. 12 Principal Components of the Ravenspurn North Concrete Platform (Courtesy of Ove Arup & Partners)

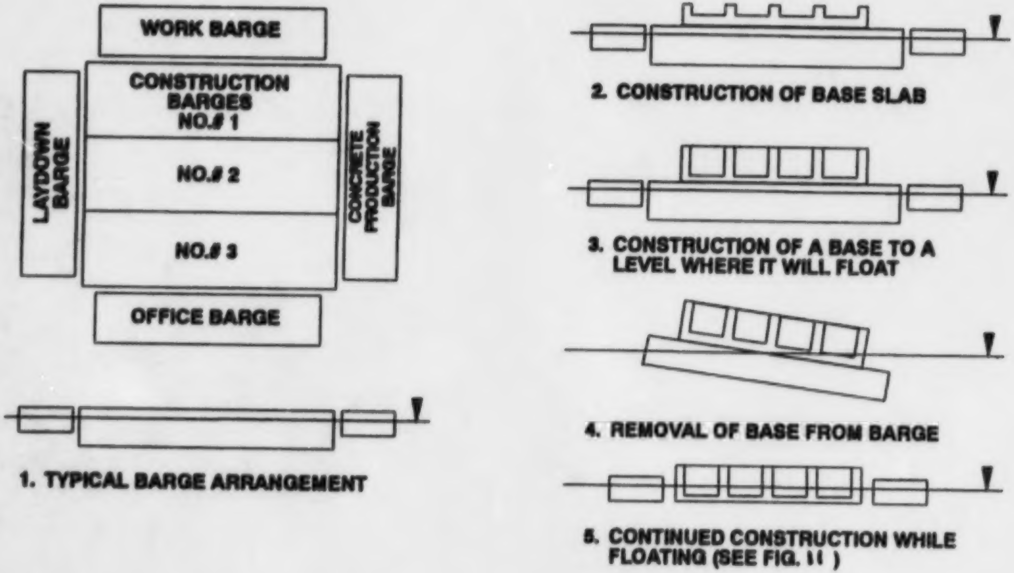

WORK BARGE
CONSTRUCTION BARGES NO.# 1
NO.# 2
NO.# 3
LAYDOWN BARGE
CONCRETE PRODUCTION BARGE
OFFICE BARGE
1. TYPICAL BARGE ARRANGEMENT
2. CONSTRUCTION OF BASE SLAB
3. CONSTRUCTION OF A BASE TO A LEVEL WHERE IT WILL FLOAT
4. REMOVAL OF BASE FROM BARGE
5. CONTINUED CONSTRUCTION WHILE FLOATING (SEE FIG. 11)

Fig. 13. Barge Construction Scenario

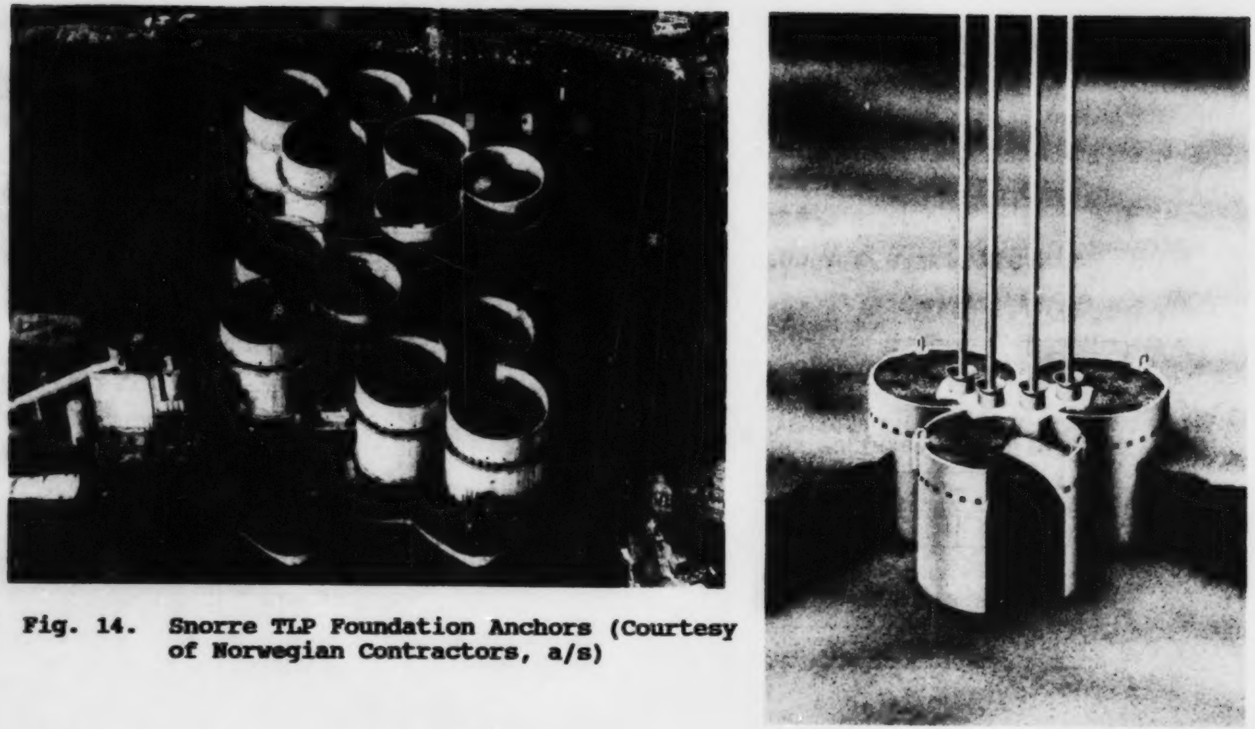


Fig. 14. Snorre TLP Foundation Anchors (Courtesy of Norwegian Contractors, a/s)

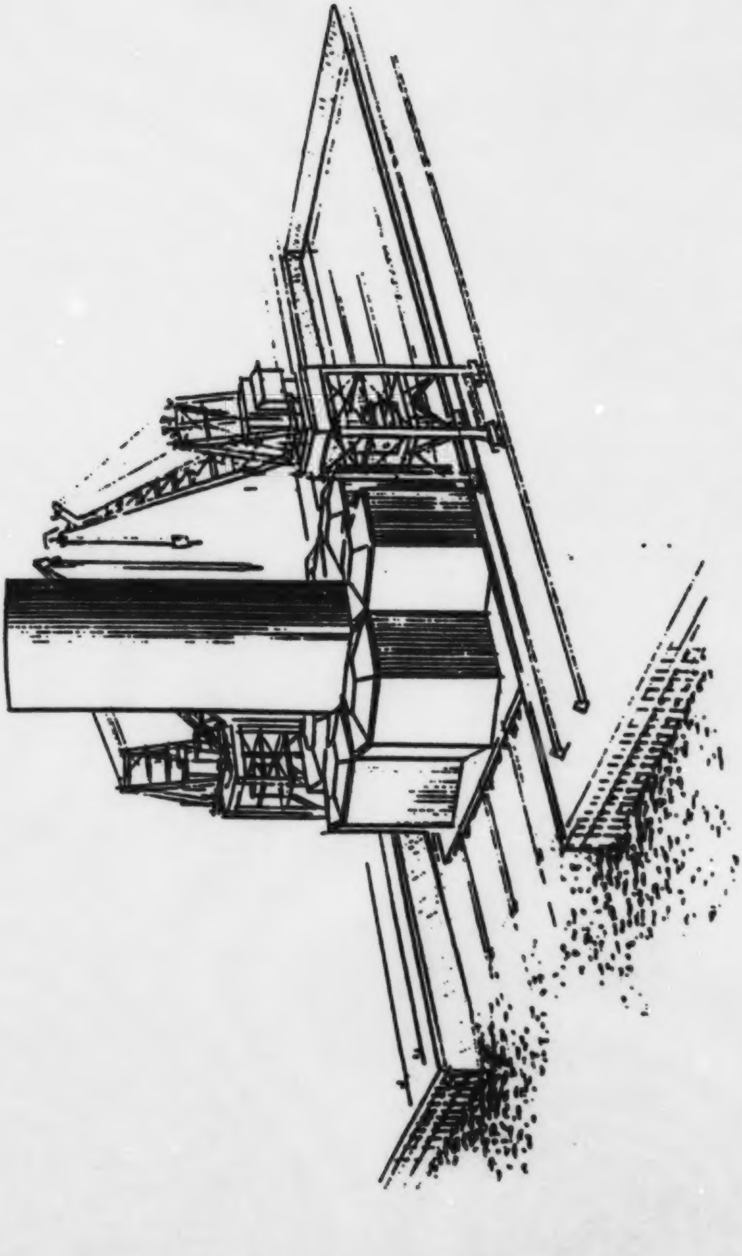


Fig. 15. Skidway Construction (Courtesy of Ben C. Gerwick, Inc.)

The Importance of Microstructure in Evaluating Concrete

By Shondeep L. Sarkar

Synopsis: Interest in microstructural investigation in determining the characteristics of concrete, cement, and aggregates is becoming a focal point in research, providing information about and insight into concrete performance and its durabiity. Investigative techniques and applications, illustrated with practical examples, are discussed in detail. Topics include high-performance concrete, lightweight-aggregate and normal concrete, aggregate-alkali reaction, mineral admixtures, paste-aggregate interface in concrete, and examination of ancient concrete.

Keywords: Microstructure, durability, deterioration, lightweight-aggregate, aggregate-paste interface, high-performance concrete, SEM/EDXA, XRDA, optical microscopy, ettringite, C-S-H, CH.

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INTRODUCTION

Concrete mix composition, placing, curing, and strength development rank among concrete technologists' prime concerns. The correlation between concrete microstructure and performance however receives scant attention, interpreted as being nonfunctional or falling outside the realm of engineering properties.

This chapter aims at stimulating awareness and interest among concrete technologists into the influence of microstructure upon the long-term concrete performance. Obviously, greater insight into concrete microstructure provides a sound basis for predicting long-range performance, which also includes a mechanistic understanding of its behaviour. This discussion also includes brief descriptions of common microstructural investigation techniques.

The importance of microstructure to several different aspects of concrete technology, such as the general evaluation of concrete/aggregate quality, strength development as a function of time, durability, chemical/physical deterioration, rebar corrosion, the role of chemical[1] and mineral admixtures[2] and the development of high-performance concrete need not be overemphasized. Thus, the different facets of concrete microstructure are dealt with in very simple terms in this chapter to enable engineers to put into practice concrete microstructural investigation along with the other laboratory tests normally carried out to determine its performance characteristics and longevity. A number of publications are also cited as further reading. Significant findings and observations from the author and other sources have been included as warranted to best illustrate the state of the art in concrete microstructure.

P.K. Mehta's (1) recent book deserves special mention for its treatment of microstructure. This meritorious work systematically lays the ground work for study of concrete microstructure from hydraulic cements and aggregates to concrete structure and properties.

Micheline Moranville-Regourd of the École nationale supérieure (Cachan, France) also stands out as well for her many publications over the years underscoring the importance of microstructure to concrete technology. Other notable research contributions on concrete microstructure have come from D.M. Roy and her colleagues at the Materials Research Laboratory, Pennsylvania State University, USA; S. Diamond of Purdue University, USA; and F. Young of University of Illinois at Urbana-Champagne, USA.

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[1] High-range water-reducing admixture (HRWRA)

[2] Silica fume, slag, fly ash, natural pozzolans

Fundamental Concrete Microstructure

In several respects, concrete resembles hydrated cement paste (HCP). After all, a number of concrete properties develop directly from cement hydration. Nevertheless, there exists a fine yet arbitrary boundary between the two, as in the cases of HCP and mortar, mortar and concrete. This phenomenon accounts for the variations in strength of HCP, mortar, and concrete. It similarly explains why a cement may undergo little or no deterioration under certain specific environmental conditions that significantly affect the corresponding concrete (for example, due to alkali-aggregate reactivity, AAR). An improperly cured concrete can show signs of damage over a period of time that may leave an HCP intact. A number of factors come into play in accounting for this dichotomy. For example, HCP is not subject to the aggregate shear action that concrete experiences during mixing. Internal bleeding, often associated with placing of concrete, is of little concern to HCP. The water-to-cement ratio (W/C) in HCP is relatively uniform throughout the matrix, compared to the W/C gradients commonly found in concrete. Unlike HCP, concrete is usually vibrated during placing. Shrinkage cracks tend to develop in HCP, but the aggregates in concrete restrain the phenomenon. Other differences were highlighted by Uchikawa (2).

From this, it should be evident that though the bulk of the HCP matrix in concrete may not be significantly different from that in a neat cement paste/mortar, subtle differences that are not constitutional in nature make concrete a separate material. The influence of these factors are obviously reflected in its microstructure.

Admittedly, the present state of knowledge of HCP microstructure outstrips that of concrete. Furthermore, all the above-mentioned factors must be taken into account when dealing with concrete microstructure. Accordingly, reference will be made only to those HCP microstructure research findings that bear most directly on concrete.

In preparation for discussing the details of concrete microstructure, the next section presents the principles underlying today's common microstructural techniques and their capabilities.

MICROSTRUCTURAL INVESTIGATIVE TECHNIQUES

Several characterization techniques, which can be divided into two broad categories, are used to study concrete microstructure.

a. Direct observation techniques, namely optical microscopy and electron microscopy, the latter includes scanning (SEM) and transmission modes (TEM).

b. Indirect interpretative techniques, mainly comprising mercury porosimetry, X-ray diffraction analysis (XRDA), and thermal methods, such as differential thermal analysis (DTA) thermogravimetry (TG), and differential thermogravimetry (DTG).

Direct Observation Techniques

Optical Microscopy

Optical microscopy remains a highly useful, simple, and inexpensive tool for the concrete technologist.

While optical microscopy offers a relatively rapid means of diagnosing concrete, sample preparation is elaborate, time-consuming, and somewhat demanding. Routine use requires adequate sample preparation facilities. Nevertheless, it can reliably serve to identify aggregates and other concrete components. Although it can provide some definite answers, additional testing, if available, is strongly recommended.

An optical microscope in a concrete laboratory is normally used for determining the W/C ratio, air entrainment (3), and for evaluating the mix proportioning (coarse and fine aggregate content) (4). These can be quantitatively determined by point counting or linear traverse methods.

Proper microscopic examination of concrete can also provide information on uniformity of composition and compaction, as well as the degree of cement hydration. Segregation or over vibration can also be detected.

Whereas, a simple binocular stereological microscope can suffice to study the above features in polished concrete slabs, a more sophisticated version called the petrological (polarized-light) microscope, provides other valuable information about the concrete.

Of late, there has been a growing concern about AAR damage to concrete. The petrological microscope provides conclusive evidence as to the presence/absence of AAR. Reaction rims, polygonization of aggregates (Fig. 1), discoloration rims, and other AAR features are clearly discernable under the optical microscope.

It also reveals the carbonation front in concretes, that usually appears either as a bright birefringent film or a rim (Fig. 2).

The petrological microscope utilizes cross-polarization of light for mineral identification (5). It provides qualitative data on the state of the aggregate in concrete, such as the presence of weathered or altered minerals (Figs. 3 and 4), and aggregate size/shape, which can affect certain properties.

For petrographic examination with the polarized-light microscope, thin concrete sections are studied in the transmitted light mode; the reflected mode is more suited to polished samples. Most modern petrological microscopes, however, come with both light sources. Preparing the thin sections demands skillfulness, whereas polishing involves impregnating the surface with resin under vacuum, followed by polishing on laps of varying fineness. Polishing, the simpler of the two techniques, nevertheless is rather time-consuming.

Those interested in a more detailed account are referred to references (9), (10), and (11).

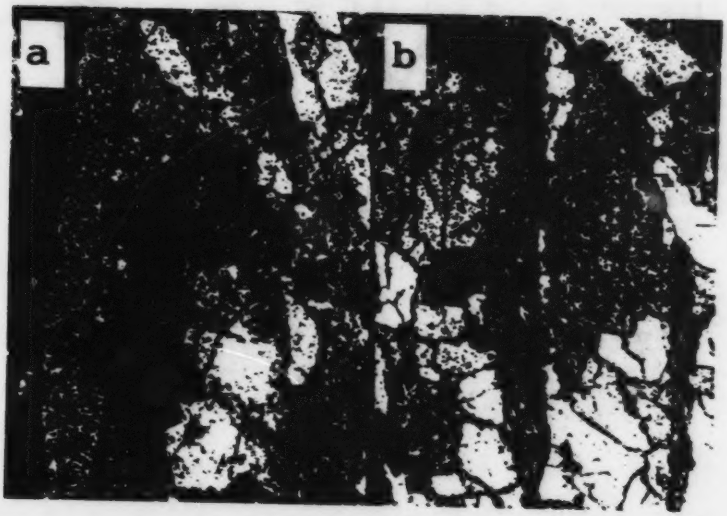

Fig. 1. A concrete showing (a) reaction rim and (b) polygonization of aggregate due to AAR

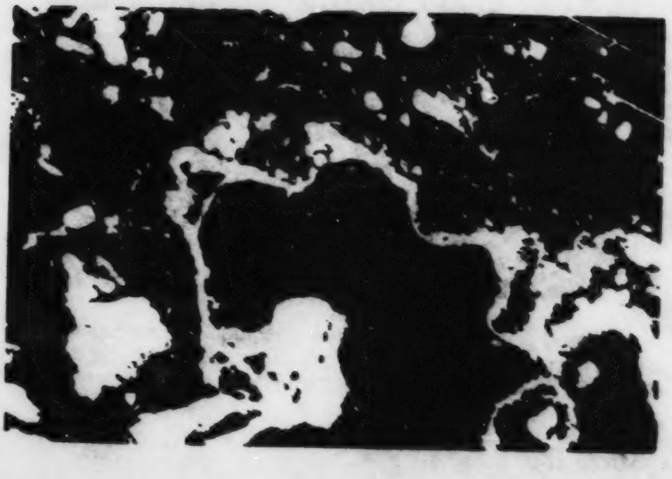

Fig. 2. Bright-coloured carbonation rim in a deteriorated concrete as seen under a petrological microscope (6)

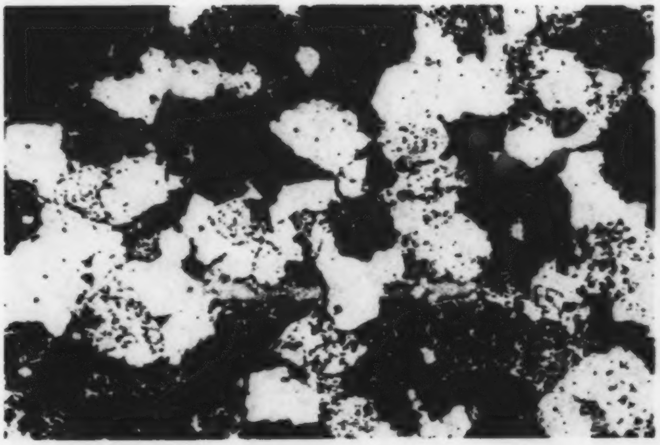

Fig. 3. A dolomitic limestone comprising well-cemented, medium-size dolomite rhombs. This aggregate was used in the concrete for the CN Tower, Toronto, Canada (7)

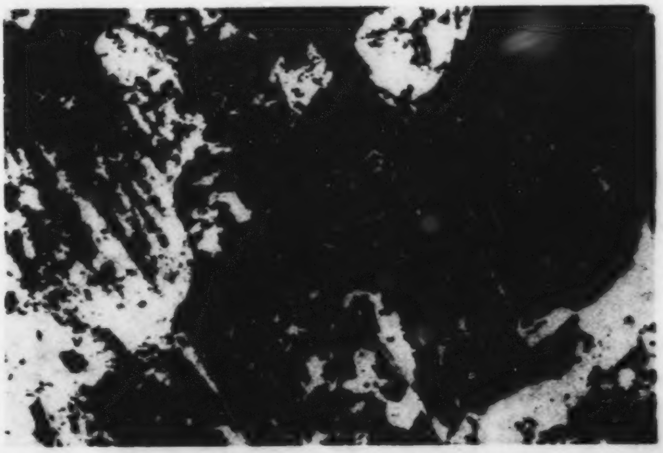

Fig. 4 A weathered granite showing feldspar alteration. An attempt to make high-performance concrete (8) with this aggregate failed due to substantial mineral alteration in the aggregates

Electron Microscopy

With its resolution limitations and inability to derive any chemical information, the optical microscope is giving way to the electron microscope (EM) attached to an energy dispersive X-ray analyzer (EDXA).

Basically, the EM is an electron-column instrument in which a narrow electron beam strikes the specimen. In the case of the transmission electron microscope (TEM), the beam is transmitted through the specimen onto a photographic plate/film. An intermediate viewing screen enables the operator to observe the features in the field of view before recording them photographically. An EDXA attachment also allows simultaneous chemical analysis of the features being viewed.

Despite its ultra high resolution, TEM is very rarely used for concrete microstructural studies, primarily because it is specifically designed to examine particles under 1 μm in size, which allows proper beam transmission through the specimen. Such fine particles are rare in concrete, except in some special types, such as silica-fume concrete. Besides, sample preparation involves grinding the sample to a very fine powder, which destroys the in situ structure. This, in turn, precludes the study of interfaces and direct phase associations. Though used extensively by some to study cement hydration, the method has yet to gain popularity for concrete microstructure examination for the reasons stated above. Nevertheless, the author and his colleague (12) have found it to be an extremely useful technique for determining the dissolution rate of silica fume in very-high-strength concrete (see Fig. 5).

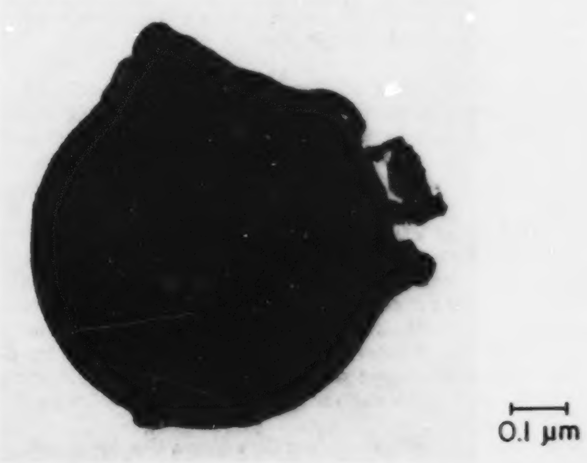

Fig. 5 A silica-fume particle in a state of dissolution in a high-strength concrete viewed under the TEM (12)

A scanning electron microscope (SEM) uses the same primary electron beam to scan the specimen surface. The secondary electrons thus emitted are converted into a scanning/TV image that can be viewed and photographed on a cathode-ray tube (CRT) screen (13). SEM allows direct observation of surface topography. Very low magnification (x20) to very high (x 10^5) can be selected. SEM provides a much greater depth of field than optical microscopes, creating a 3-D effect. In addition, SEM imagery is easy to interpret, especially in combination with EDXA analysis (14). Figure 6 provides an example of a scanning electron micrograph of concrete and its corresponding EDX spectra obtained from different components of interest.

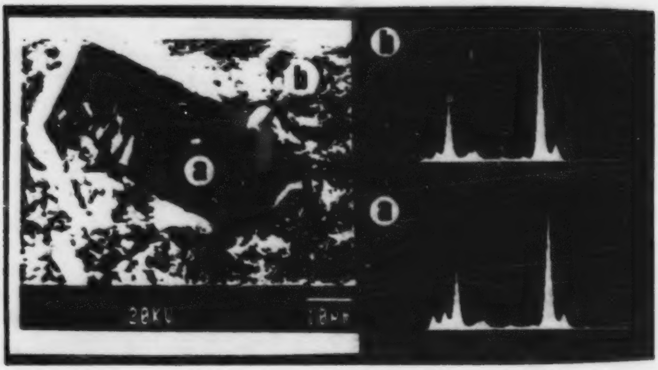

Fig. 6 A slag grain, part of which has hydrated, surrounded by C-S-H in a concrete. EDX spectrum represents, the unhydrated portion (a), and the surrounding C-S-H composition (b)

One of SEM's great advantages lies in the possibility of examining fractured concrete specimens, although flat polished specimen can also be observed with relative ease. Sample preparation of fractured surfaces requires minimal effort, and the sample can be made electrically conductive with only a thin coating of Au, Au-Pd, Cr, or C without destroying the inherent structure.

The third major tool used (in conjunction with either SEM or TEM) is the energy dispersive X-ray analyzer (EDXA). It consists of either Li-doped Si or Ge solid-state detector maintained under cryogenic conditions. The X-ray photons emerging from the specimen are collected by the detector and initially converted (with statistical variation) into voltage pulse output through a complex electronic process. This is then transformed into counts/channel that are displayed on a CRT screen. Due to its characteristic energy position, each element can be readily identified by means of elemental cursors included in the software.

Si-Li detectors with a standard Be window thickness of 6-7μm, however, absorb long wavelength radiations, thus preventing the identification of elements with $Z < 11$, that is, Na. Nevertheless, ultra-thin window Si-Li, diamond, and Ge detectors now available, though at slightly increased price, enable detection of elements down to C ($Z = 4$). This is of particular significance to concrete technologists, especially those involved in the microstructural study of carbonation, in which the morphological distinction between CH and $CaC0_3$ is often difficult to ascertain. Both minerals yield only the characteristic Ca peak at 3.68 KeV on the EDX spectrum with the standard detector, whereas C and 0_2 peak identification (derived from $CaC0_3$) is possible with the ultra-thin detector (Fig. 7). As a matter of interest, this may be the only microstructural confirmation of carbonation if the concrete happens to contain limestone ($CaC0_3$) as coarse aggregate. Here, even other microstructural techniques such as X-ray diffraction analysis (XRDA) and differential thermal analysis (DTA) may fail to distinguish carbonation from limestone aggregate in the bulk specimen.

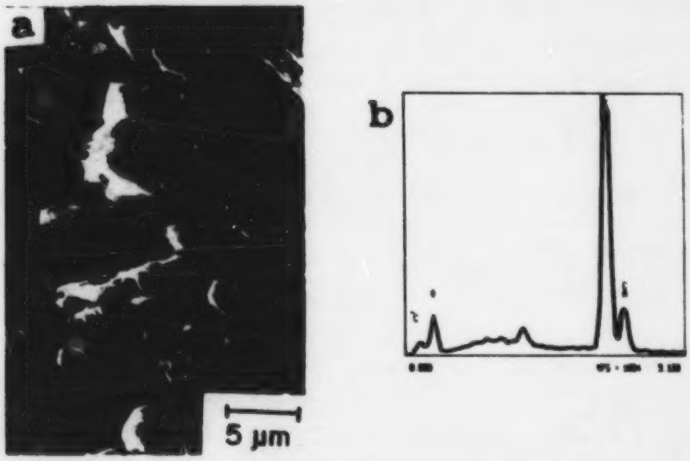

Fig. 7 (a) $CaC0_3$ crystal formation due to carbonation in a concrete. (b) The carbon and oxygen peaks in the corresponding EDX spectrum confirms this as $CaC0_3$

Chemical analysis by ultra-thin window detector alone allows distinction between the needle-like morphology of ettringite* and thaumasite,** which may be present in chemically deteriorated concrete.

Since EDXA analysis cannot be restricted to an arbitrarily small volume, spurious contributions as a result of the beam passing through the crystal being analyzed (especially if they are thin platy or fine fibrous types, such as ettringite or monosulfate) may jeopardize accurate analysis. Silicon peak may appear in the EDX spectrum of ettringite from underlying or neighbouring C-S-H. To positively distinguish ettringite from thaumasite (Si being a constituent element of the latter), the presence or absence of C in the spectrum must be determined. This is only possible if the EDXA unit is equipped with an ultra-thin window detector.

The ZAF+ software now available with most EDX systems allows quantitative phase analysis on polished samples, though phase delineation can be a major problem. To overcome this, backscattered electron imaging (BEI) is often used (15). This technique utilizes another type of electron that is emitted from the specimen in proportion to atomic number.

In practical terms, however, concrete technologists chiefly interested in performance characteristics of concrete or the effects of deterioration will require only phase identification, morphology, distribution pattern, or association with other phases in a concrete to determine cause and effect. Carefully examining the fracture surface eminently fulfills this need.

---

* $\{Ca_6 [Al(0H)_6]_2 \cdot 24\ H_20\}\ [(S0_4)_3 \cdot 1\ 1/2\ H_20]$

** $[Ca_3 \cdot Si(0H)_6 \cdot 24\ H_20]\ (C0_3)\ (S0_4)$

+ ZAF = atomic number (Z), absorption (A), and fluorescence (F) correction factors

Indirect Interpretative Techniques

This section deals briefly with the three most prominent techniques commonlyused in microstructural investigation.

Mercury Porosimetry

While providing authentic, useful information regarding the pore size, pore distribution, and total concrete porosity, mercury porosimetry does not truly represent concrete microstructure. It can, however, be related indirectly to microstructure (16) and serves as a good supplementary tool for detailed microstructural determinations with emphasis on porosity.

Thermo-Analytical Techniques

These techniques are widely used to identify and estimate the hydration phases in cement pastes and concrete (17). The most extensively used techniques are based on differential thermal analysis (DTA) or thermogravimetric analysis (TG). The derivative of TG (DTG) is also used to provide a better deliniation of peaks.

In DTA, the loss of water, decomposition reactions, or crystallization transitions are determined through endothermal or exothermal effects. In many instances, the DTG provides more accurate quantitative information. One of the major advantages of these techniques is that compounds such as sulfoaluminates, calcium hydroxide, gypsum and calcium carbonate can be identified relatively quickly. The amorphous or near amorphous phases that cannot be easily identified by XRD can be determined by thermal techniques.

Overlapping of peaks due to oxidation effects are possible. In many instances the analysis should also be carried out in inert atmosphere to circumvent this problem. If a reaction is very slow then it may not register a peak in thermal analysis. Although very useful data are obtained by thermal analysis techniques it becomes necessary that they should be confirmed using other methods of investigation.

X-ray Diffraction Analysis (XRDA)

XRD specimens are finely ground powders which allow random orientation. The technique uses the principle of characteristic planar reflections from a multitude of randomly oriented crystals that are recorded on a chart recorder (18). These reflections, which appear as peaks, can be matched to standard JCPDS+ XRD patterns. Modern units integrate the complete JCPDS computer file comprising over 33 000 inorganic/organic compounds. Automatic/manual search-and-match programs are available for looking through operator-created subdirectories containing specific information, such as the likely mineral phases in concrete. This enhancement has greatly alleviated painstaking, tedious physical comparison.

+ JCPDS = Joint Committee on Powder Diffraction Standards

XRD is a proven adjunct technique for the concrete technologist, especially since it can provide accurate phase identification. Interpreting results with concrete constituents evolve around the innumerable peak overlaps, and the strong masking effect generated by the presence of coarse and fine aggregates. Even if the coarse aggregates are removed during sample preparation (a technique frequently used by the author), some finer rock fragments tend to remain, together with the fine aggregates (normally quartz [SiO_2] and the feldspar [Na/K/Ca alumino-silicate] group). This creates a permanent but unavoidable masking effect in the diffraction pattern as illustrated in Figure 8.

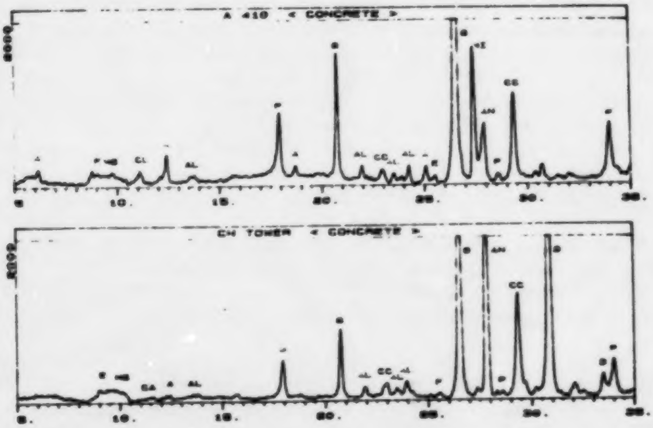

Fig. 8 XRD traces of two concretes showing the strong peaks of quartz and feldspar minerals. E = ettringite, MS = monosulfate, CL = chloroaluminate, CA = carboaluminate, AL = aluminohydrate, P = portlandite (CH), A = albite (feldspar), MI = microcline (feldspar), AN = anorthrite (feldspar), Q = quartz, CC = calcite, D = dolomite

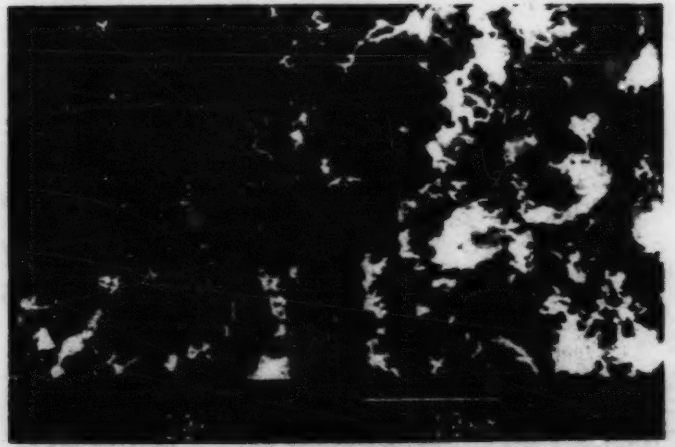

Fig. 9 Showing fibrous type I C- S-H

The region of interest of C_3S and C_2S hydration products lies between 25° and 35°2θ. This, unfortunately, coincides with the strongest peak of quartz (26.69°2θ). feldspar minerals (between 26° and 34° 2θ), and calcite (29.38°2θ). Besides, though XRDA is capable of identifying crystalline forms of chemical compounds and minerals, it cannot efficiently identify C-S-H or other amorphous phases. They appear in the diffraction pattern as broad peaks or humps. Interpreting specific XRD results of a particular phase among many, requires caution, care, and thought.

The region between 4° and 20°2θ, however, allows identification of some common concrete phases, including the characteristically strong peak of CH at 18.10°2θ, devoid of any peak overlap. The other identifiable peak of CH, often overlapping, occurs at 34.08°2θ. Ettringite and thaumasite peaks, appearing in proximity at 9.09°2θ and 9.15°2θ, respectively, can also be distinguished with sufficient care. Other readily identifiable phases in this region are monosulfate[1] (9.93°2θ), chloroaluminate[2] (11.37°2θ), carboaluminate[3] (11.70°2θ), and different forms of aluminohydrate. Figure 8 depicts some of these identifiable peaks. Particular conclusive identification, it must be stressed, lies with an overall matching of peaks that goes beyond finding only the most predominant.

When used to study carbonation phenomenon in concrete, XRD proves particularly useful in distinguishing the different polymorphic forms of $CaC0_3$,[*] even when SEM fails to conclusively distinguish them from morphological examination or EDXA chemical analysis; EDXA can only identify elements, not phases.

It is true to say that, among the various microstructural investigative methods available to the concrete technolgist for microstructural studies, SEM/EDXA is the most powerful and commonly used. Supportive XRD data can also be very useful in enhancing the interpretation of results. Nevertheless, additional data obtained by other interpretative techniques, such as petrographic description of a concrete from optical microscopy, porosity data from mercury porosimeter, or DTA/DTG curves can often prove equally useful.

Whatever technique is used, adequate background information about the concrete is essential, since all the particular features must be evaluated properly and collectively to arrive at conclusive results.

MICROSTRUCTURE OF CONCRETE

Introduction

The foregoing section on microstructural investigative techniques provides a glimpse at the tools researchers can rely on in characterizing concrete. This familiarization is key to grasping the value of such techniques through examining samples and interpreting the findings. This section illustrates how the concrete technologist uses these techniques to analyze some important aspects of concrete microstructure and composition.

---

[1] $[Ca_2Al(0H)_6]\ [0.5\ (S0_4) \cdot 3H_20]$

[2] $[Ca_2Al(0H)_6]\ [Cl \cdot 2H_20]$

[3] $\{Ca_6\ [Al(0H)_6]_2 \cdot 24H_20\}\ [(C0_3)_3]$

[*] Aragonite, vaterite and calcite. All or some of these forms of $CaC0_3$ may be present in concrete

Normal Concrete

Typically, the microstructure of a normal (W/C ≥ 0.50) concrete is comprised of C-S-H gel with fibrous peripheral outgrowths (Fig. 9) (which has been classified as Type I C-S-H by Diamond (19)), large platy CH crystals in the paste (Fig. 10a), oriented CH crystals at the paste-aggregate interface (Fig. 10b), monosulfate crystals (Fig. 10c), and ettringite needles (Fig. 10d).

Of these, concrete technologists are most intensely interested in the paste-aggregate interface (or transition zone), since it bears directly on concrete strength properties.

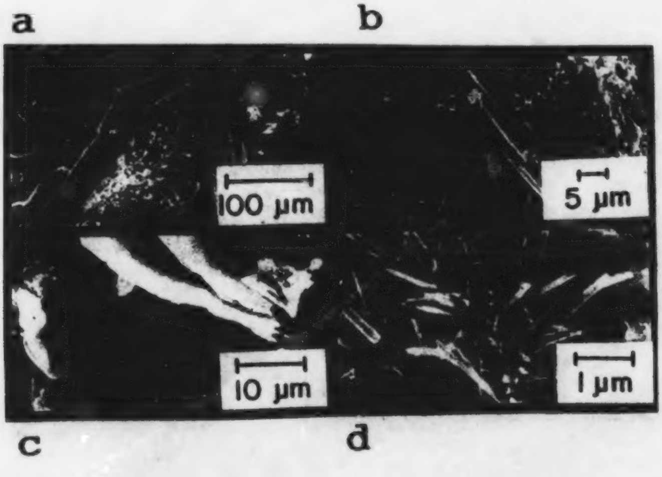

Fig. 10. Microstructure of normal concrete: (a) Large tabular CH crystals. (b) Oriented CH crystals at the paste-aggregate interface. (c) Monosulfate crystals. (d) Ettringite needles

The Transition Zone

Micromorpological research on the transition zone by Barnes et al. (20) broke new ground in concrete technology. Though limited to mortars, their research shed light on the formation of the transition zone in concrete. In fact, one might say it was a springboard for voluminous research on the subject. Their microstructural explanations treated phenomena such as the duplex film, development of oriented CH crystals (with their c-crystallographic axis parallel to the grain surface), and stacking of CH crystals in the open spaces, together with the presence of Hadley grain (hollow shell) structures at the interface.

The subject continues to attract the attention of microstructural researchers worldwide and has been the focus of two international symposia. The first (21) included several presentations on the development of bond structure, experimental methods of study, influence of time/environment, relationship between bond structure and the mechanism of deformation, and crack development.

A number of interesting papers on microstructural investigation of paste-aggregate bonding in concrete were also presented at the second symposium (22) with the main focus on the different types of interfaces that develop when admixtures are used in concrete.

Monteiro et al. (23) used SEM and XRD to demonstrate that both CH and ettringite are formed at the paste-aggregate interface as confirmed by the author (24) (see Fig. 11).

Fig. 11 CH and ettringite crystals at the paste-aggregate interface (24)

Monteiro and Mehta (25) state that the weakest component in concrete is the paste-agregate transition zone, positing that concrete strength can be enhanced by grain refinement in this zone.

Prominent debonding at the interface, which can adversely affect strength, is a prime concern of concrete technologists that is readily verifiable under SEM (Fig. 12).

Any significant departure from usual microstructural characteristics stands out, signalling possible deterioration, compositional variation (low W/C ratio), or chemical/mineral admixtures. The concrete technologist then proceeds to determine the causes and effects of such changes on concrete properties through microstructural investigations, mainly SEM/EDXA.

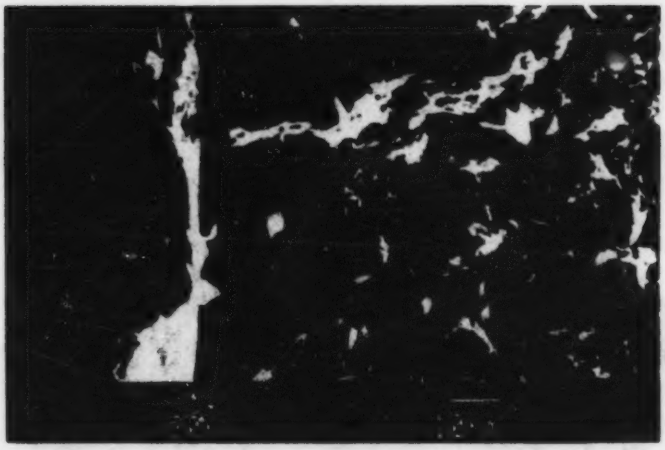

Fig. 12 Debonding at the paste-aggregate interface in a concrete. A = aggregate, P = paste

Durability and Concrete Deterioration

Concrete durability and deterioration are so closely related that they must be considered together. Indeed, durability is the main factor in appraising deterioration.

Most reports on concrete deterioration relate to marine conditions, due to the recent large-scale construction of offshore concrete oil rigs, using widely differing concrete compositions. Establishing their performance characteristics has gained high-profile status among concrete technologists, especially since microstructural data has proven to be very useful in design and construction improvements of these and other marine structures.

Mehta's outstanding overview of this topic (26) covers several microstructural features of deterioration arising from chemical reactions as well as physical factors, such as freeze-thaw and wave action. Regourd (27) has produced a worthy companion to Mehta in which she presents SEM, XRDA, and electron microprobe analysis (EMPA) results, identifying a number of deterioration products and discussing the role of the potentially deleterious ions (Mg^{2+}, $S0_4^-$ and Cl^-) that frequently occur in relatively high concentrations in seawater.

Conjeaud's article (28) eminently illustrates the mechanism of seawater attack through a series of scanning electron micrographs. The formation of ball-like structures comprising thick and long hexagonal rods of a new form of ettringite, called the Si-ettringite, was proposed in this work.

Synthesizing these research results leads to the conclusion that CH degradation can occur, either through leaching out or conversion to $CaC0_3$, which weakens the concrete. Magnesium ions can form brucite ($Mg(0H)_2$), together with some substitution in C-S-H. Secondary products, such as chloroaluminate, gypsum, ettringite, and thaumasite can develop in the paste, depending upon cement type, exposure conditions, permeability, and the quality of concrete. Mehta (26), in summarizing a number of case histories of concrete deterioration, shows that permeability is the most important criterion for concrete durability or longevity.

During the course of microstructural investigations of concrete deterioration (29-31), the author has identified a number of these products, as presented in Figure 13.

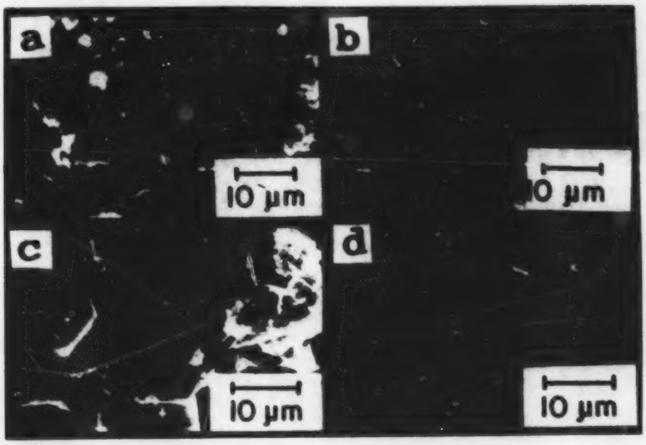

Fig. 13 Microstructure of deteriorated concrete: (a) A carbonated layer containing brucite formed on a concrete surface exposed to Arctic marine conditions for 3 years (29). (b) Profusion of secondary ettringite in a severely deteriorated concrete. (c) Chloroaluminate crystals in a deteriorated concrete median barrier (31). (d) Increased porosity in a deteriorated surface of a concrete parapet due to leaching of CH and fine aggregates (30).

Ettringite is ubiquitous in concrete, albeit in varying levels. Although often mistakenly identified as the main deteriorative agent, ettringite, according to Mehta (26), has seldom been conclusively proven to be the sole cause of cracking in concrete. He attributes this to the presence of chloride ions which inhibit the expansive phenomenon associated with ettringite.

XRD, SEM, and mercury porosimetry studies into the durability of concrete subjected to simulated Arctic marine conditions has recently shown that Cl^- ions are often localized in the 10-mm surface layer; the concentration of Mg^{2+} in C-S-H is rather small (32).

A recent microstructural investigation by Novak and Colville (33) of degraded concrete foundations in California revealed the wide spread presence of some rare, efflorescent salts, such as mirabelite[1], thenardite[2] , darapskite[3] , nahcolite[4], trona[5], halite[6], and birunite[7], in addition to ettringite, thaumasite, calcite, and gypsum. Though a competent X-ray diffractionist can distinguish these by XRDA, they are often recognizable under the SEM (see Fig. 14).

---

[1] $Na_2S0_4 \cdot 10\ H_20$

[2] Na_2S0_4

[3] $Na_2\ (N0_3)\ (S0_4) \cdot H_20$

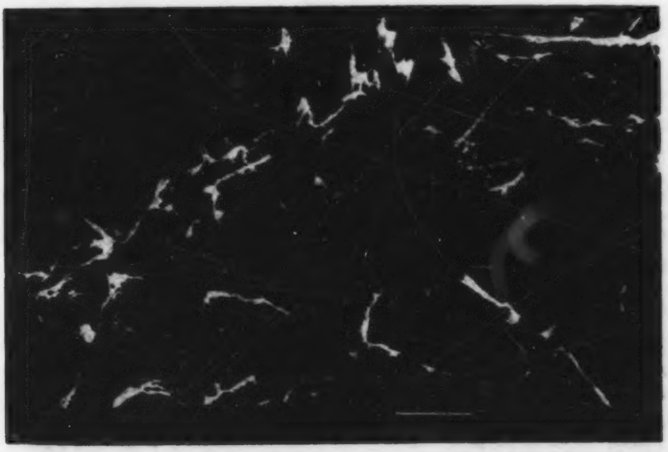

Fig. 14 Glauberite[8] (efflorescent salt) in a deteriorated mortar. Thenardite or mirabelite occurring in the same specimen (6)

Microscopic examination of weathered concretes (34) shows that pores and cracks tend fill with CH, alkali-silica gel, and ettringite. Correlation between cracking and corrosion in reinforced concrete determined by SEM by Francois and Arliguie (35) showed tht paste-aggregate interface is damaged in the tensile zone, leading to increased permeability.

Propagation of cracks is said to be another symptom of deterioration. Hornain and Regourd (36) used stereological technique to quantify the network of cracks in concrete. They classified cracks and fissures into three types depending on length (centimeter, millimeter, and micrometer). They found these respective cracks to occur at the paste-aggregate (coarse and fine) interface, in C-S-H, CH, and sulfoaluminate. Carrasquillo (37), on the other hand, claims that X-ray radiography is a powerful tool for observing internal microcracks in concrete. Other investigators (34) prefer using fluorescent dyes to fill up the cracks for a clearer image under the microscope.

Carbonation can also have deleterious effects on concrete. Though the standard phenolphthalein test is used most commonly, damage assessment depends on microstructural investigation, either by optical microscopy (Fig. 2), SEM, or XRDA. Some of the commonly sighted $CaC0_3$ morphologies are shown in Figure 15.

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[4] $NaHC0_3$

[5] $Na_3H(C0_3)_2 \cdot 2H_20$

[6] NaCl

[7] $8.5\,(CaSi0_3) \cdot 9.5\,(CaC0_3) \cdot CaS0_4\ CaS0_4 \cdot 15\,H_20$

[8] $Na_2Ca\,(S0_4)_2$

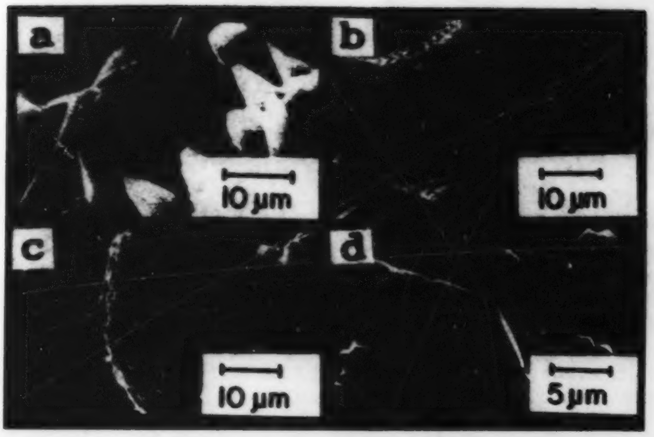

Fig. 15 Morphology of $CaCO_3$ minerals: (a) Dog-tooth. (b) Floral distribution. (c) Globular pattern. (d) Stacked

Roy's review article (38) clearly emphasizes the role of microstructural factors in physical degradation and chemical attack of cement paste and concrete.

Rebar Corrosion

In the mass of literature on concrete, little has been reported on the microstructural characteristics of rebar corrosion, despite the fact that is often key to deterioration. Indeed, reinforced concrete failures probably stem from rebar corrosion. Marchese's (39) microstructural study of rebar corrosion offers several elegant micrographs of corrosion products with explanations of their possible modes of formation. Figg's list (40) of the Fe-oxides, hydroxides, and Fe-chloro complexes that can form due to corrosion has proven very useful to X-ray diffractionists in identifying corrosion products.

Glasser and Sagoe-Crentsil (41) used TEM/SEM examination of corroded concrete to propose a sequence of Fe-oxide and hydroxide formation. SEM research by Koelliker (42) revealed that the superficial skins in corroded concrete comprise $CaCO_3$.

A recent microstructural study by the author (31) confirmed the presence of $CaCO_3$ crystals at the rebar-paste interface. The lamellar and globular types of corrosion products (39,40), the latter enriched in Cl^- ions (Fig. 16) as well as multilayer corrosion (41) were also identified.

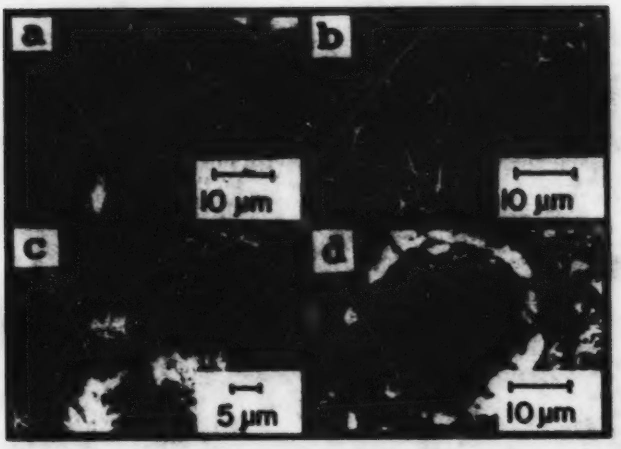

Fig. 16 (a) Well crystallized tabular Fe-rich crystals. (b) Lamellar crystals of Fe-oxide or hydroxide. (c) Globular mass that has burst open to expose a floral arrangement. (d) Globular mass rich in Cl^- with delicate whiskery growths.

Micromorphological investigation of corrosion products can also embrace the effectiveness of corrosion inhibitors. The changes in composition and morphology at the interface can help clarify the role of an inhibitor product in reducing corrosion. For example, Tashiro's (43) SEM results proved that zinc oxide as an admixture is a very effective corrosion inhibitor.

Fibre-reinforced Concrete

Reinforcement in concrete is not limited to steel bars. A variety of fibres has figured in many testing program, including steel, glass, polypropylene, and asbestos, to confer strength to fresh and hardened concrete.

Fibres are, however, mainly used to control cracks and as secondary reinforcement. Compared to ordinary concrete, fibre-reinforced concrete features an additional transition zone in the region of the reinforcing fibres, whose microstructure is different from the paste (43). The nature and composition of the fresh paste influences the transition zone in two ways:

(i) air entrapment around the reinforcement and

(ii) inefficient packing of very fine cement particles in the 20-40 μm zone around the fibre interface.

Bentur and Mindess (44) demonstrated that this initially water-rich zone tends to be porous, preventing the development of a microstructure as dense as that of the paste. Its considerable CH content (Fig. 17) results in a weak matrix/fibre link, as confirmed by Wei et al. (45). In comparing the microstructure of monofilament fibre (steel) reinforcement and bundle fibres (glass), Bentur (46) observed that concretes made with the latter embrittleness due to the growth of CH crystals between the filaments occurs. Limited formation of hydration products due to less penetration of cement particles within these spaces

(Fig. 17) in some glass fibres indicates less affinity for cement paste than does asbestos.

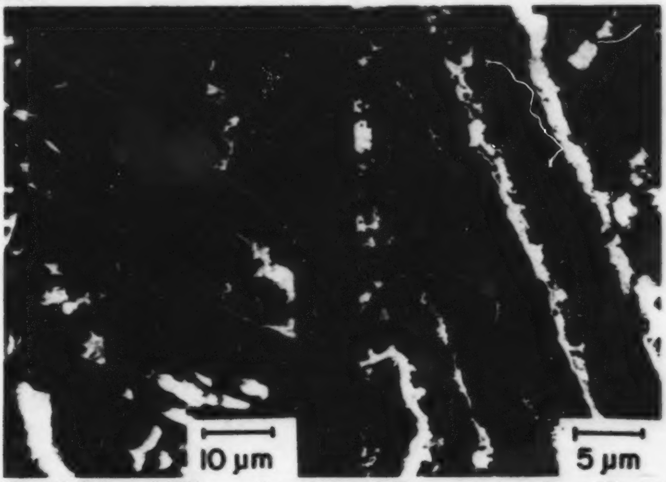

Fig. 17 (a) Oriented CH crystals at the steel fibre-paste interface (29). (b) Vacant spaces between fibrils in a glass-fibre reinforced concrete

The author in the course of his investigations also noted another interesting feature comprised of CH which closely resembles the vesicular structure of volcanic rocks. This phenomenon probably results from local concentrations of water at the interface.

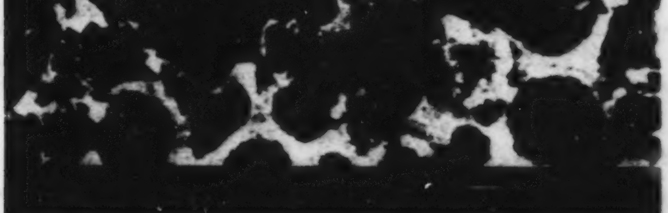

Fig. 18. Vesicular structure at the fibre-paste interface

Mills and others (47,48), however, have shown that alkali-resistant glass fibres tend to exhibit surfacial nucleation and growth of CH crystals.

Ancient Concretes

Microstructural examination of ancient concretes can provide valuable insight into why some modern concretes fail. It also opens a window onto concrete technology in the past as the following examples demonstrate.

Idorn and Thaulow (49) carried out microstructural investigation of Portland Hall (Gravesend, Kent), built in 1847 by W. Aspdin, son of J. Aspdin. This highly interesting study concluded that the concrete, which is still in good condition, was made from a coarse ground cement at low W/C ratio, was well compacted, cured properly, and air-entrained.

The HCP of an 84-year-old reinforced concrete bridge revealed main hydration products of CH, fibrous C-S-H, and carboaluminate (50). The concrete had not been adversely affected even after such a long service life.

Cole et al. (51), who studied the microstructure of a 40-year-old concrete dam in Australia, however, reported the presence of a zeolitic type of reaction product in addition to precipitated amorphous alkali-silicate gel. Grudemo (52) who carried out investigation of a concrete wall built in 1914 in a water tunnel at the Porjus hydel power station, reported the practical difficulties associated with XRD analysis of concrete. However, C-S-H, monosulfate and CH were detectable.

Lightweight Aggregate Concrete

The demand for lightweight aggregates as a structural material is on the increase for economic and practical reasons. For example, its light weight translates into considerable reductions in foundation bulk and into easier handling of precast elements (53). Besides, its use enables the construction engineer to lay a thicker cover on the reinforcing steel for better protection. Higher thermal insulation can also be achieved due to its lower density and cellular structure.

Although several types are commercially available (clinker; sintered, pulverized fuel ash; foamed slag; expanded clay; shale/slate; pumice; diatomite, wood particles; expanded perlite; and vermiculite), all share nearly identical basic characteristics. All are much less dense than natural aggregates. From the mechanical point of view, their grain surfaces have a rough texture (Fig. 19) that affords better bonding with HCP.

Porous in nature (Fig. 20), the grains absorb water. According to Swamy and Lambert (54), who carried out a study correlating microstructure and pore structure of Lytag (pulverized fuel ash), the porosity can vary between 25 and 75%, depending on the raw materials, manufacturing process, and final aggregate size. The absorbed water is then yielded to the surrounding matrix, promoting "inner curring".

In general, the inherent internal/external structure of lightweight agggregate has given LWA concretes a proven track record under severe exposure conditions (55).

The most important microstructural criterion of lightweight aggregate lies in studying the aggregate-paste interface, usually characterized by excellent bonding (Fig. 20).

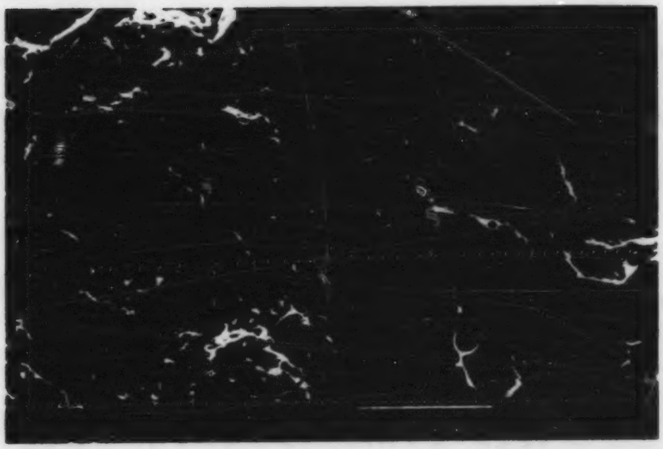

Fig.19 Rough grain surface of lightweight aggregates

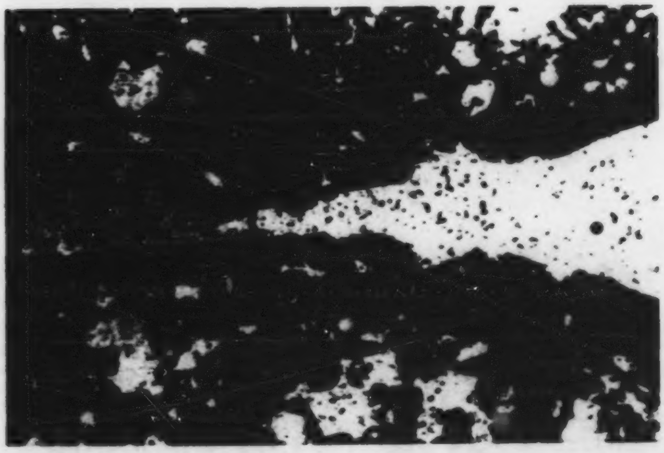

Fig. 20 Strong bonding between paste and lightweight aggregate in a concrete. The porous nature of the aggregate can also be seen

Holm et al. (56), who investigated a number of existing mature lightweight concrete bridge decks as well as the *USS Selma*, a lightweight concrete ship constructed in 1919, have stressed the influence of aggregate type on bonding. Khokorin's (57) report, based on several Russian investigations, suggested that bonding results from chemical and mechanical interaction. Khokorin further demonstrated that the permeability of lightweight concrete is equal to or even lower than normal concrete.

Though pozzolanic nature of lightweight aggregate is purported to contribute to higher concrete strength, a recent investigation by Zhang and Gjorv (58) points to a negligible level of reactivity with cement paste.

A further study by these authors (59), demonstrated, through a series of elemental X-ray maps and BSEI, that lightweight aggregates with dense outer shells behave very similarly to normal aggregate: that is, CH can form at the paste-aggregate interface. On the other hand, the porous exterior of lightweight aggregate promotes denser, more homogeneous bonding.

At this point, it should be obvious that concrete technologists investigating lightweight concrete should equally divide their attention on aggregate microstructure (that is, the nature of its exterior and interior) and the paste-aggregate interfacial zone in order to determine their influence on the engineering properties of lightweight concrete.

Alkali-aggregate Reaction (AAR) in Concrete

AAR in concrete remains a formidable hurdle, despite the worldwide concerted efforts of scientists and technologists. Extensive research is being conducted on reducing, even preventing, AAR. Regular symposia (such as the 8 international conferences on alkali-aggregate reactions) and workshops are held across the world to discuss the problem of AAR in concrete.

Microstructural investigative techniques, such as optical and scanning electron microscopy, are very often effective tools for examining affected concrete and aggregate, identifying the reaction products, and determining the degree of AAR damage caused.

The expansive reaction phenomena, divided into (a) alkali-silica (b) alkali-silicate, and (c) alkali-carbonate reactions, the influence of environmental conditions, moisture content, and the role of aggregates have been exhaustively discussed in the literature (60). Recommended reading includes the last two conference proceedings on alkali-aggregate reactions (61,62).

Microstructurally, AAR damage appears in aggregate polygonization, reaction rim (Figs. 1a and b), exudation as amorphous and crystalline gel, and peripheral discoloration of aggregate (Fig. 21). To this effect, some outstanding research results were reported by Regourd and Hornain (63). Shayan's recent re-examination of AAR in concrete (64) also presents a series of scanning electron micrographs of reaction products that are very similar to those cited earlier by Regourd (63).

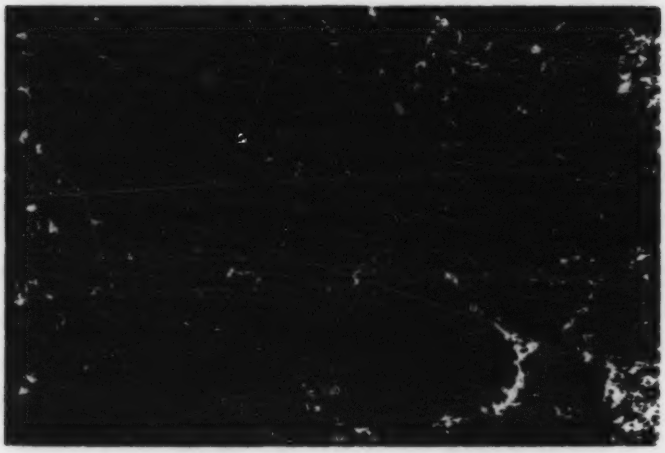

Fig. 21 Peripheral discoloration of an aggregate due to AAR

The Modern Trend: High-performance Concrete (HPC)

Concrete technologists nowadays are converging on developing HPC. Enhancements to concrete quality are continually sought after, not just for increased compressive strength, but for the inherent greater durability and for economic reasons. HPC is usually made by replacing part of the cement with supplementary cementitious materials, such as slag, silica fume, fly ash, or a combination thereof. Matrix densification is achieved through high dosages of HRWRA, allowing drastically reduced W/C ratios. It is interesting to note that the practical limit of HPC or high-strength concrete (HSC) as it was formerly called, is approaching the 150-MPa mark (65).

Evidently, high-tech research into HPC is proceeding on a worldwide scale. Needless to say, HPC microstructural studies are of key importance, especially for designers, engineers, and architects. Realizing the full potential of this range of materials rests on controlling and defining its characteristics, and, whenever necessary, introducing changes in composition. In other words, it depends on microstructure.

The microstructure of high-strength cement paste systems (hot-pressed, autoclaved, impregnated, and low W/C) has been described by Regourd (66). She also examined the crystallinity and composition of cement hydrates, microporosity, cracking phenomena, and fracture paths. However, as stated previously in *Fundamental Concrete Microstructure* section, subtle yet definite differences do exist between the microstructures of HCP, mortar, and concrete. So, one needs to microstructurally examine HPC in order to understand more fully its long-term behaviour and correlate the microstructural characteristics with strength development properties.

Rosenberg and Gaidis (67) proposed a mechanism (partly based on microstructural investigation) for strength enhancement in high-strength concrete containing a dispersion of silica fume in water and an additive to provide mix stability and proper viscosity.

Beaudoin and Feldman (68) point to porosity and pore geometry, C-S-H composition, composite behaviour, and fracture as the important factors influencing the strength parameter in high-strength cement paste systems. It must be noted that all these factors are either directy or indirectly related to the system's microstructure. Over the last few years, the author's laboratory has focused on HPC microstructure, producing a number of pertinent articles (8,12,24,69-72). In addition, the author and his colleagues have been regularly monitoring the state of a high-strength silica-fume field concrete with SEM and other methods (73,74). Their microstructural findings reveal the presence of some small CH crystals and ettringite needles in the HCP of this concrete, together with a slightly porous skin at age 4 years. A slight reduction in compressive strength was also noted.

Comparison of these studies has revealed that HPC presents a different type of paste-aggregate interface, is characterized by the absence of oriented CH crystals (Fig. 22).

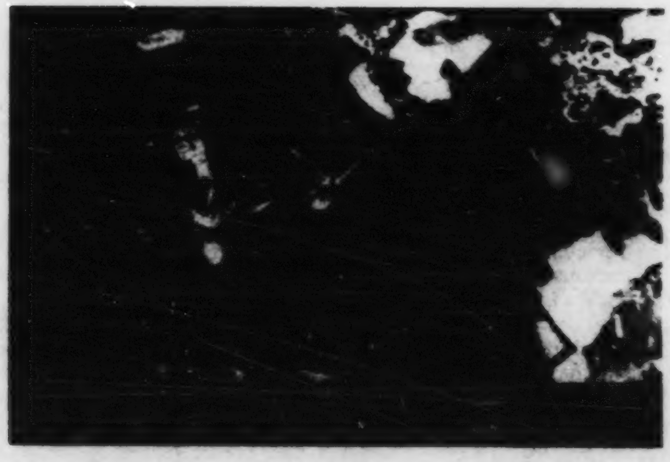

Fig. 22 The paste-aggregate interface in a high-strength concrete. Note the absence of oriented CH crystals

BEI and normal secondary electron imaging of the transition zone confirms that significant improvement in the aggregate-matrix bonding is achieved when silica fume is used: the transition zone transforms into a dense mass (24,75) similar to the HCP. Combining silica fume with low W/C ratio in making HPC also results in a dense paste, mostly comprising amorphous C-S-H gel (Fig. 23).

Fig. 23 Dense paste in a HPC (W/C+S = 0.25) containing silica fume

The microstructure and pore characteristics of concrete with different levels of silica fume addition (2.5 - 10%) was also described by Marusin (76).

The principle governing the pozzolanic reaction of slica fume with cement was first reported by Grutzeck et al. (77). They amply demonstrated, through SEM microstructural examination, that silica fume rapidly dissolved a lime-saturated solution and that CH is eventually consumed in this context. When trying to extrapolate this model to HPC microstructure, the writer (12) observed a retardation effect as a result of the low W/C+S normally used.

The review article by Cohen and Klitsikas (78) on the correlatin of hydration mechanism and strength development as a function of silica fume in cement composites is noteworthy in this regard.

Besides silica fume, slag stands as an important mineral admixture for concrete. Though considerable research has been done on the properties of slag cement, the microstructure of concrete containing slag, especially HPC, has yet to be extensively investigated. Nevertheless, the review article (79) on the structure and behaviour of slag-cement hydrates is an extremely erudite background reading. Detwiler et al. (80) have shown that the transition zone of concrete can be significantly improved with a 30% cement replacement by slag. The synergistic role of slag in enhancing the late-age strength in a slag/silica-fume concrete was determined from SEM/EDXA (71). Unlike silica fume particles that are ultrafine (< 1 µm in diameter), the finite size of slag grains makes it possible to identify them under the SEM, even when they are partially hydrated as shown in Figure 6.

The relation between slag sand hydration and mechanical properties of concrete investigated microsturally by Nagataki et al. (81) indicates loss of tensile strength and embrittlement of concrete.

Considering that slag is known for its latent hydraulicity, alkali activation of slag has lately received considerable renewed attention (82). The present state and potential of alkali-activated slag concretes has been elaboratedy discussed by Talling and Brandstetr (83).

A recent microstructural study of alkali-activated slag concrete by the author (84) has shown the presence of a dense amorphous gel, though cracked at times to constitute the HCP (Fig. 24), also observed by Malopszy and Petri (85). It has been postulated that the cracks are directly related to the dosage of water glass used to activate the slag.

Fig. 24 Dense but cracked gel in an alkali-activated slag concrete (85)

The third mineral admixture used in concrete is fly ash. Its beneficial effects are now well established (86). Though the hydration and pozzolanic activity of fly ash in concrete cement paste has been closely examined from the microstructural angle by numerous researchers, notably Diamond (87), Uchikawa (88), Marusin (89), and Tenoutasse and Marion (90), the role of fly ash in HPC has not been fully studied. Scali et al. (91), who demonstrated increases in compressive strength and resistivity, used SEM to evaluate the effect of fly ash on concrete engineering properties. They observed a more open structure in fly ash concrtete.

Recently, it has been shown (72) that the presence of high-alkali fly ash in HPC yields a sudden strength spurt at a very early age (12 h). Microstructural examination suggests this result from the rapid dissolution of alkalies from the fly ash surface. Like its counterparts, silica fume and slag, fly ash added to HPC results in a dense microstructure with unhydrated or partly reacted particles, distinctly visible at the microscopic level. Some of the features that develop in fly-ash HPC are shown in Figure 25.

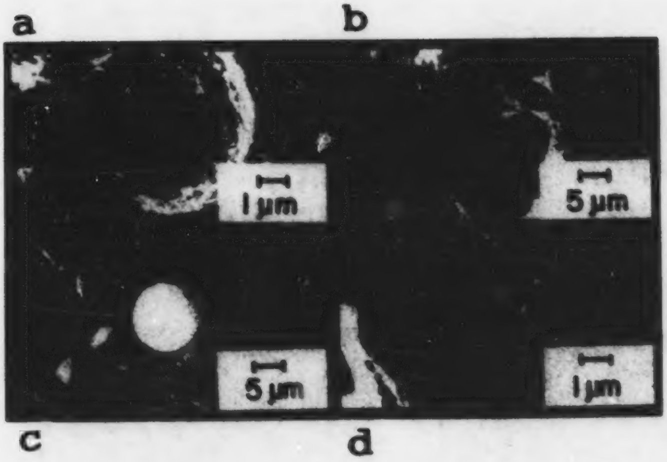

Fig. 25 (a) An unreacted fly-ash particle; (b) A partly reacted fly-ash particle; (c) Liesengang ring structure (C-S-H located radially around fly ash) and (d) Stacked CH association with fly ash

CONCLUSION

To date, concrete technologists have relied on engineering tests to determine the quality, durability, and other properties of concrete. While these methods will remain valuable tools, increasing attention is focusing on the advantages of microstructural investigation in providing strategic information in correlating concrete engineering properties to the durability and the cause/effect of concrete failure and deterioration. As has been demonstrated by numerous researchers, microstructural examination yields more accurate data in terms of pinpointing the exact cause of failure, or in predicting the long-term performance of concrete.

Although a number of microstructural investigative techniques exist, SEM/EDXA stand out as the quintessential instrument in microstructural characterization for ease of use and interpretation, high resolution, phase association, morphological information, and, perhaps most importantly, the simultaneous elemental analysis for specific phase identification. This can be complemented by optical microscopy and XRDA for petrological identification of aggregates and mineralogical identification of hydrated phases.

It must be remembered that concrete, mortar, and hydrated cement paste have differences that can be readily revealed using microstructural techniques. Accordingly, characterizing concrete may provide results that differ from that of the corresponding mortar or cement paste. One example of this is the paste-aggregate interface, an important feature that occurs only in concrete. Analyzing the mortar or cement paste will shed very little light on interface behavior.

Long-term performance and determining the cause of deterioration are of major interest to concrete technologists. Microstructural examination can also be used as a window on ancient concreting practises, providing valuable insight into performance, as well as serving to identify corrosion products, such as determining whether rebar corrosion in specific cases is due to chloride or carbonation.

As for lightweight aggregate concrete, microscopic techniques have proven extremely useful in studying the paste-aggregate bond, the most important parameter affecting strength. This has led to the discovery that LWAs with dense outer shells perform similarly to normal-weight aggregates, while promoting denser bonding with the paste.

The fascinating development of high-strength (or high-performance) concrete is likewise proving a fertile ground for microstructural investigation, which features enhanced durability and impermeability as well as outstanding strength. Since mineral admixtures, such as fly ash, blast-furnace slag, and silica fume, are basically required to produce HSC, their microstructural behavior in concrete is of prime interest in predicting long-term performance and characteristics.

As research into this exciting new field of concrete technology gains momentum, scientists and technologists will focus increasing attention on the various ways that understanding concrete, cement, and aggregate can ultimately play a crucial role in delimiting the characteristics and performance.

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Role of Fracture Mechanics in Concrete Technology

By S. P. Shah and M. A. Tasdemir

Synopsis : The application of fracture mechanics to concrete has provided new ways of understanding of the material behavior during fracturing. In this paper, the concepts of linear elastic fracture mechanics are summarized. It is shown that a single fracture parameter, such as the critical stress intensity factor, cannot be simply applied to quasi-brittle materials such as concrete. This inapplicability of conventional fracture mechanics is due to fracture process zone and precritical crack growth. In order to determine the size-independent fracture parameters, nonlinear type of fracture mechanics models based on observations of fracture processes using laser holography as well as acoustic emission measurements are presented. Crack propagation and damage distribution in cement-based materials containing fibers are also examined by quantitative image analysis and laser holographic interferometry. Based on fracture mechanism observed during experimental studies, an R-curve approach is presented to predict toughening of matrices due to fiber reinforcement. The theoretical predictions show good agreement with the experimental results.

Keywords: Concrete; crack growth; crack propagation; fibres; fracture mechanics; image analysis; laser holographic; toughening.

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INTRODUCTION

The field of fracture mechanics originated with the classical work of Griffith in the early 1920s on the brittle fracture of glass. Failures in various types of structures such as pressure vessels, airplanes, ships and pipelines, led Orowan and Irwin to extend Griffith's theory to deal with other materials which were not completely brittle. Fracture mechanics concepts have been very useful to predict brittle fracture and fatigue failure of metalic structures (1,2). Considerable development has occurred in the last thirty years vaulting in modification of Griffith's ideas and new concepts to account for the ductile behavior of metals. As a result of these efforts, standard testing techniques became avaliable for obtaining fracture mechanics parameters for metals and desining based on these parameters are included in relevant specifications (1). Fracture mechanics is gaining acceptance for use in quasi-brittle materials such as concrete, rock and ceramics.

Structural engineering expands and changes its scope following new needs and new technologies. Modern computer-aided analysis and use of concrete for special structures such as reactor containment vessels, floating marine structures and storage tanks have led to growing interests in the cracking behavior of concrete (1,3,4).

Kaplan (5) was the first researcher to apply linear elastic fracture mechanics (LEFM) to concrete. Since then, fracture of quasi-brittle materials such as cement-based composites has been studied for three decades, however, fracture mechanics has not been really introduced in the design of concrete structures, because most of the past work was based on the concept of LEFM which does not completely describe crack propagation in concrete (6,7). In the determination of fracture toughness (K_{IC}) by using LEFM, a significant effect of size and geometry of specimen was observed (7). Impossibility of obtaining a valid fracture toughness parameter from LEFM and laboratory-sized specimens was pointed out by Shah and McGarry (8). Following this, several researchers have addressed this problem and all aspects of the problem have been extensively discussed in recent symposia and technical committees (1,9 to 18).

Recently, succesful approaches have been made to use fracture mechanics concepts in design. For example, size effect observed in shear failure of reinforced concrete beams can be predicted using fracture mechanics based models (14,19 to 24). Succesful use of fracture mechanics has also been demonstrated by Ingraffea, Sauoma, Linsbauer et al. for predicting crack stability in dams (25 to 27). There is a growing interest in the field. Draft recommendations have been proposed : to determine fracture energy using a notched beam test (28), determination of fracture parameters (K^{s}_{IC} and $CTOD_c$) of plain concrete using three-point bend tests (29) and size-effect method for determining fracture energy and process zone size of concrete (30). Round-robin tests are in progress to determine the effects of geometry and the testing system on the fracture mechanics parameters. Studies to determine the influence of test set-up and geometry of specimens on various materials parameters are being conducted by various technical committees.

The aim of this paper is to provide the basic information about fracture mechanics of concrete and to stimulate further research into experimental, theoretical and numerical studies in the field.

FRACTURE MECHANICS CONSIDERATION

A.A.Griffith was the first to make a quantitative connection between the critical normal stress and crack size (31 to 33). He suggested that all materials contain flaws or defects that have the effects of stress concentration in highly localized regions (34). A pre-existing crack can propagate when the elastic energy released during the crack extension is greater than the surface energy needed to create the new surfaces. Griffith derived his equation for glass, which is a very brittle material. Irwin and Orowan extended independently Griffith's theory to metals and then, Irwin connected the global concept of Griffith with a local crack tip parameter (32).

The stress at a crack tip

A crack in a solid can be stressed in three different modes as illustrated in Figure 1. Normal stress give rise to the opening mode or Mode I loading. The crack face displacements are perpendicular to the crack plane (Fig.1a). In-plane shear results in mode II which is called sliding mode; crack surface displacements are in the crack plane and perpendicular to the leading edge of the crack (Fig.1b.). The tearing mode or Mode III is caused by out-of-plane shear. Crack surface displacements are in the crack plane and parallel to the leading edge of the crack (Fig.1c.).

In the mode I case, crack tip stresses, σ_{ij} can be expressed as :

$$\sigma_{ij} = \frac{K_I}{\sqrt{2\pi r}} f_{ij}(\Theta) \ldots\ldots\ldots\ldots\ldots \qquad (1)$$

where σ_{ij} are the stresses acting on a material element d_x d_y at a distance r from the crack plane, and $f_{ij}(\Theta)$ are known functions of Θ. Since K_I describes all the crack tip stresses it is an important parameter, called the stress intensity factor (SIF). The subscript I denotes mode I (35). Similar solutions are obtained for other modes with SIFs K_{II} and K_{III} with different functions of the variable Θ. Note that K has dimensions of "stress x $\sqrt{\text{length}}$" whereas the stress concentration factor is dimensionless (35 to 38). The strain energy release rate which is conventionally given by the symbol G in honor of Griffith. In mode I case, the critical strain energy release rate G_{IC} which can be determined by measuring the stress σ_c required to fracture a plate with a crack size of 2a, Fig.2. G_{IC} may be expressed in terms of the critial stress intensity factor K_{IC} as :

$$G_{IC} = \frac{K^2_{IC}}{E} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

Since G_{IC} is closely related to K_{IC}, the size-dependency of G_{IC} is essentially the same as that of K_{IC}. On the other hand, a good fracture parameter should be, independent of specimen size which will be disussed later.

APPLICATION OF FRACTURE MECHANICS TO CONCRETE

Even though concrete is a quasi-brittle material, its fracture toughness is currently not used as a design tool. Since concrete structures are usually reinforced, the designer often neglects the tensile strength of plain concrete (39). But a structural engineer needs to build reliable structures and increase efficiency and accuracy of design. This led researchers to define and evaluate the fracture toughness of concrete (40). Investigations on concrete fracture carried out so far, concentrate on the failure under pure mode I loading. Some of them can be seen in references (1,3 to 18).

A common test method to evaluate fracture toughness consists of using a notched-beam specimen which is centrally loaded as shown in Fig.3. K_{IC} can be calculated from knowledge of the maximum load, initial (precast or sawn) notch-length "a_o", and the appropriate equation for determining K_I. If this is done for concrete, than a large dependence on the depth of the beam and size of the initial flaw is observed. This is shown in Fig. 3 (41). As a result of such observation, researchers have concluded that it is difficult to obtain a unique material parameter such as K_{IC} to quantify fracture toughness of concrete from laboratory-size specimens (8,11,42).

Another reason for the inapplicability of linear elastic fracture mechanics (LEFM) to concrete can be demonstrated by examining the result of the notched-beam tests. If the recorded load-crack mouth opening displacement (CMOD) is essentially linear up to the peak load, then one can apply LEFM. A typical result for a notched mortar beam is shown in Fig.4 (42). Substantial nonlinearity prior to the peak load is observed. This nonlinearity is due to stable, precritical crack growth.This precritical growth is considerable, as depicted in Fig.5.In this figure,precritical crack growth observed by using a dye penetrant technique is sketched (43). The length of the precritial crack length is different along the width of the specimen. This means that the surface crack measurements are not adequate to describe the fracture process zone (44).

The trend shown in Fig. 6 implies that high strength concrete is more tough which is contrary to the generally accepted observation that high-strength concretes are more brittle. If fracture toughness is expressed using the definition and test methods developed for metal, then it is observed that K_{IC} increases with compressive strength as shown in Fig. 6 (45). Thus, there is a need to define a more appropriate fracture toughness for concrete. Use of both K_{IC} and the critical crack tip opening displacement ($CTOD_C$) achieves a more realistic model, which correctly predicts that high-strength concrete is more brittle than normal concrete (46). This will be explained later.

The existence of a mode II failure in concrete has also been investigated (47,48). There is at present little information avaliable about mode III failure in concrete (31,49).

Applications of fracture mechanics may bring along significant benefits. It will be possible to introduce new designs and utilize new cement based materilas which have been developed recently and exhibit considerable high toughness values. Fracture mechanics approach may be useful in some applications such as: i) cracking in massive unreinforced concrete structures, ii) impact resistance, crack propagation at high strain rates, iii) finite element analysis for serviceability, iv) modeling of shear tractions along a rough propagating crack in cement based composites, v) combined mode fracture of concrete under compression-shear and tension-shear type of loadings, vi) concrete structures of unusually large sizes, vii) characterizing new materials in concrete construction such as macro-defect free cement composites, and very high strength concrete, viii) studies of time and temperature effects on the crack propagation in concrete under different loading conditions. It can be expected that, in the twenty-first century the rapid development of new technologies will require more sophisticated approaches in design.

Mixed Mode Fracture Studies Under Compression

Crack initation and propagation in rectangular blocks of mortar with both central notches (open crack) and central closed crack (stone-matrix interfacial cracks) under combined Mode I and Mode II were studied at Nortwestern University. In these studies, specimens shown in Fig.7 were made of mortar with a 1:3:0.6 mixture (cement:sand:water by weight). Specimens were cast in plexiglass molds as described in references 50 to 54. Four different crack orientation angles (18°, 36°, 54° and 72°) with respect to the loading direction were used. Specimens were 152 mm wide, 203 mm high and 76 mm thick. Separate end blocks were cast to eliminate end effects. For open crack a 2 mm thick and 50.8 mm long steel blade and for closed cracks a 7.6 mm wide and 50.8 mm long piece of limestone were placed vertically through the thickness of the specimen during casting, and embedded in the mortar. The steel blade was removed 4 1/2 hours after casting, for easy removal. The stone pieces were left in the specimens.

To obtain kink initiation angles, for the open crack case, the maximum hoop stress theory (MHST) proposed by Erdogan and Sih (55) and for the closed aggregate-matrix interface crack the $\max K_I(\Theta)$ criterion proposed by Steiff (56) were used. As shown in Figure 7, there is a good agreement between experimental and theoretical results. For the open crack case a similar trend was obtained by Tirosh and Catz (57) for the material PMMA ($\rho/a=0.031$) as depicted in Fig.7. Further explanations and applications of other mixed mode crack initiation theories to this problem are discussed in references 51 and 53. It was observed that the mixed mode crack propagation as well as mode I crack propagation can be predicted by modifying LEFM theories given in references 50,51 and 53 for both open and closed crack cases. The maximum hoop stress theory was applicable to concrete and the K_I SIF at the tip of the propagating crack was dominant, provided corrections were done by applying normal and shear tractions to match those observed experimentally at the various crack stages. This will be shown below.

FRACTURE PROCESS ZONE AND DETERMINATIONS OF FRACTURE MECHANICS PARAMETERS

For plain concrete, to study the fracture processes at the microscopic level, two noninvasive techniques are currently being studied at Nortwestern University. These two techniques are acoustic emision source location and identification methods and laser holography. Some pertinent results from these studies will be described in this section. The details of these techniques are given in references 58 to 60.

Acoustic Emission (AE)

Acoustic emissions are microseismic activities originating from within the test specimen when subjected to an external loading. Acoustic emissions are caused by local disturbances such as microcracking. An acoustic signal travels to 4 piezoelctric transducers which convert the acoustic signals (mechanical wave forms) to electric signals. The electric signals are captured by a digital oscilloscope. The time of arrival of the signal at each transducer depends on the distance of the transducer from the AE source. The source, frequency and amplitude of the AE events have been used to quantify the nature of microfracture in various materials. AE sources are determined by calculating the difference in time taken for the wave to arrive at different transducers. Velocity of the waves in the specimen was determined by an ultrasonic system. To find the rate of AE events, an oscilloscope is connected to an IBM-AT by a GPIB interface bus, Fig.8 (58).

Some typical results from the AE source location studies are shown in Fig.9, Single-edged-notched specimens of mortar were tested in a closed-loop testing machine. The crack mouth opening displacement (CMOD) was used as a feedback control. In Fig.9, load vs. CMOD curves of a specimen is given. The location of AE events observed during the different stages of load-CMOD response are also given along with microscopically observed surface crack length. The following conclussions can be summarized from the study: i) Microcracking localized before peak load and the microcrack cluster progresses forward with increasing CMOD, ii) Few events occur ahead of the microscopically observed crack tip but events continue to occur far behind the crack tip indicating closing pressure due to ligament connection, iii) Rate of AE events reach a maximum immediately after peak load, iv) Effective crack length estimated from the modified LEFM approach (to be described later) was consistent with optical and AE measurements (38,59).

Laser holography

When two holograms of the object at two levels of loading are superimposed (sandwich holograms), fringes are observed. The number of fringes give the displacement along the sensitivity vector. It is possible to observe crack profiles with an accuracy of about 1/2 micron, as the crack propagates with an increase in load.

Typical crack profiles for a (center cracked plate) mortar specimen are shown in Fig.10. Note that the radius of the loading hole was 12.7 mm and the length of the notch was 22.2 mm. Fig.10 shows that the holographically measured crack profiles are narrower than those predicted by LEFM .This implies that bridging tractions exist along the cracks in quasi-brittle materials (60).

A nonlinear fracture model

AE measurements revealed a substantial energy consumption behind the optically observed crack tip. Crack profiles observed using laser holography indicated pinching of the crack tip as compared to an elastic crack profile. These and other observations indicate that a single fracture parameter may not be sufficient to describe fracture processes occuring in these laboratory size specimens. Several nonlinear fracture models have been proposed to occount for the nonlinear fracture process zone(42,61 to 64). A model proposed by Jenq and Shah is briefly described in this paper.

Generalized fracture mechanics model for concrete-- In tension-shear type of loading, for the two-dimensional case of a crack subjected to shear and normal tractions, which may be generated through fiber-bridging or aggregate-interlock effects, with nonzero SIFs at the crack tip and assuming linear elasticity, the strain energy release rate, G, can be expressed as

$$G = \frac{K^2_I}{E} + \frac{K^2_{II}}{E} + \int_o^{CTOD} \sigma(w)dw + \int_o^{CTSD} \tau(s)ds....(3)$$

K_I = net SIF for mode I,
K_{II} = net SIF for mode II,
$\sigma(w)$= tensile traction acting on the crack surface,
CTOD= crack tip opening displacement,
$\tau(s)$= shear traction acting on the crack surface,
CTSD= crack tip sliding displacement.

In the present derivation, the energy is assumed to dissipate along the crack. Eq. 3 contains most of the basic fracture parameters (i.e. K_I, K_{II}, CTOD, CTSD) that might be involved in a general failure condition. In Eq.(3), the shear traction (τ) is a function of the sliding displacement, friction coefficient (μ) and possibly, crack opening displacement. The effect of the shear traction on the structural system is similar to the slip type of plastic deformation in metals. Therefore, the contribution of energy by shear traction will not be negligible and the global mechanical behavior may not be predicted just based on energy criteria if the shear traction is not negligible.

It should be noted that if no tractions exist along the crack surface, Eq.(3) will reduce to the Griffith crack, i.e.

$$G = J = \frac{K^2_I}{E} + \frac{K^2_{II}}{E} \quad \ldots\ldots\ldots\ldots\ldots \quad (4)$$

However, if one assumed that there were no stress concentration (K_I and K_{II} are equal to zero), then Dougdale-Barenblatt type model will be as follows:

$$G = J = \int_0^{CTOD} \sigma(w)dw + \int_0^{CTSD} \tau(s)ds \quad \ldots\ldots\ldots \quad (5)$$

The original idea of the traction is attributed to the atomic type of force in Dugdale-Barenblatt model which is not obtained from the global uniaxial stress-strain or stress-separation relationship.

To examine the validity of Eq.(3) and the associated fracture parameters, it was first decided to simplify the studied problems so that the influence of shear tractions were ignored. The simplest fracture problem of this type is encountered in mode I failure of plain concrete.The effect of tensile traction can be simulated by an effective Griffith crack approach.The Jenq-Shah two parameter model is based on effective elastic crack approach .These two fracture parameters are the critical stress intensity factor (K^s_{IC}) and the critical crack tip opening displacement ($CTOD_c$) (Fig.11) which is used to quantify the stable crack growth prior to peak load.

Two parameter fracture model (TPFM)

The extent of stable precritical crack growth (Stage II Fig. 11) can be substantial for concrete. It has been observed that when the K_{IC}, is calculated ignoring this precritical crack growth,(if the initial notch length a_o, is used in the evaluation of K_{IC}) the value of fracture toughness (K_{IC}) depends on the geometry and dimensions of specimens (42,46).

An effective Griffith crack model has been proposed by Jenq and Shah to incorporate the precritical crack growth (42,46). It is assumed that a crack of initial length "a_o" will critically propagate when the SIF at the tip of the effective crack "a" reaches the critical value K^*_{IC}. The superscript in K^*_{IC} denotes inclusion of prepeak stable crack growth. The effective crack "a" is defined such that the crack tip opening displacement at the tip of the original crack "a_o" reaches a critical value $CTOD_c$ shown in Fig.11. One can determined these parameters from pertinent LEFM equations and the measured values of initial compliance (C_i in Fig.11) and unloading compliance (C_u in the same figure) at or near the maximum load as described next.

In general, LEFM yields the following equations :

$$K_I = \sigma\sqrt{\pi a}\; F_1\ (\alpha) \quad \ldots\ldots\ldots\ldots\ldots \quad (6)$$

$$CMOD = 4\sigma\ a/E.V_1\ (\alpha) \quad \ldots\ldots\ldots\ldots\ldots \quad (7)$$

$$CTOD = CMOD.Z\ (\alpha,\beta) \quad \ldots\ldots\ldots\ldots\ldots \quad (8)$$

Where σ = applied stress, E=Young's modulus, D=depth of specimen, CMOD=crack mouth opening displacement, CTOD=crack tip opening displacement, $\alpha=a/D$=notch-dept ratio, $a=a_o+\Delta a_p$, a_o=initial notch length, Δa_p=pre-peak crack extension, $\beta=a_o/a$, V_1 and Z are geometry correction factors. Knowing the initial compliance (C_i in Fig.11) and using Eq.7 and $a=a_o$, the value of Young's modulus E can be determined. By substituting this value of E in Eq.7, an effective crack length a is calculated such that the compliance obtained using Eq.7 is equal to the measured elastic compliance at the peak load (C_u in Fig.11). K^*_{IC} and $CTOD_c$ are the calculated at peak load using Eqs.(6)-(8) and the effective crack length "a". Thus, the two fracture parameters, K^*_{IC} and $CTOD_c$ (as well as the Young's modulus E) can be determined from a single notched beam test.

If the fracture parameters K^s_{IC} and $CTOD_c$ and Young's modulus E are known, the peak stress and peak load of any structure can be calculated using Eqs. (6)-(8). Combining Eqs. (6)-(8) to :

$$K_I = \frac{CTOD.E.\sqrt{\pi}}{4\sqrt{a}} \frac{F_1(\alpha)}{V_1(\alpha)Z(\alpha,\beta)} \quad \ldots\ldots \quad (9)$$

The value of the effective "a" at the critical load is determined from Eq.(9) for a given set of values of K^s_{IC} and $CTOD_c$. Then using Eq.(6) the corresponding stress value at the critical load can be determined.

Based on tests of three different sizes of single edge notched beam specimens, Shah (38) showed that the two fracture parameters K^s_{IC} and $CTOD_c$ thus determined are nearly size-independent (Fig.12). The two parameter fracture model requires the use of a closed loop testing system with CMOD such a feedback signal control to obtain stable load response. But both the fracture parameters K^s_{IC}, $CTOD_c$ and E can be obtained from a single test (29,42,46). TPFM was validated recently by Alvarado et al. (65), Miller et al. (60) and Llorca et al. (66), based on mode I fracture study using either a plate specimen with a central notch and a point load applied at the center of the notch or a single edge notched beam specimen.

Knowing the fracture parameters K^s_{IC} and $CTOD_c$, and Young's modulus, E the peak load for the laboratory size specimens mentioned in the previous paragraph can be predicted by a simple iterative procedure using LEFM equations similar to Eqs. (6)-(9). Note that this model is valid for both notched ($a_0 \neq 0$) and unnotched ($a_0 = 0$) specimens. Combining K^s_{IC}, $CTOD_c$ and E, Jenq and Shah (46) have obtained a material parameter Q, which is similar to the characteristic length, proposed by Hillerborg et al. (67). Q can be expressed as :

$$Q = \left[\frac{E.CTOD_c}{K^s_{IC}} \right]^2 \quad \ldots\ldots\ldots\ldots \quad (10)$$

Jenq and Shah (42,46,68) and John and Shah (70) have used this material parameter to predict: i) tensile strength;

ii) split-cylinder strength; iii) size effect on K_{IC}, iv) size effect on modulus of rupture; v) strain rate effect; and vi) size effect in tension-shear type of mixed mode fracture and shear.

The transition between relatively large size structure and small size specimens has been modeled by Bazant and his colleagues by a size effect law (64,69).

It is interesting to note that the critical crack extension ($\Delta a_p = a - a_0$) calculated using the two fracture parameters (K^*_{IC} and $CTOD_c$) gives a good indication of relative fracture toughness of materials. For example, critial crack extension decreases with an increase in compressive strength (Fig.13). This means that for very high strength concrete, the precritical crack growth tends to be negligible and as a result, the material behavior becomes more classically brittle. The same observation can be made with respect to the effect of increasing strain rate. It was observed that the higher the strain rate, the lower the value of Δa_p (Fig.14). Thus, the extent of critical crack extension (which can be ascertained from the two fracture parameters) appear to be a useful index for toughness for quasi-brittle materials. For a classically brittle material, $\Delta a_p = 0$ and LEFM is applicable. For concrete, Δa_p can be significant and as a result, one fracture parameter is not adequate. The value of Δa_p will depend on mixture composition, strain rate, size and geometry of the structure.

Fracture Process Zone Studies Under Compression

It is necessary to see if an extension of the Mode I model could be possibly used for the compression-shear type of mixed mode loading where a tensile opening of the crack faces is also accompanied by sliding.

Types of specimens described above are shown in Fig.7. For the open crack case double exposure holograms were made with the set up that given in references 51,54 and 71 at different loading stages so that each hologram contained displacement information between two subsequent loading stages. These were inspected at the end of the test to obtain crack opening and sliding data. For both open and closed crack cases, two specimens of each group were tested by an automated holography system (Fig.15). It was possible to make a large number of holograms by this system and observe the propagation of cracks at different stages of loading. For the open crack case, a typical fringe pattern for $\beta=54°$ is shown in Fig. 16. Various stages of kink crack development for a specimen with an aggregate-matrix interface notch inclined at angle $\beta=36°$ are shown in Fig.17. It is seen from such crack patterns that the debonding at the stone matrix interface always preceeds the initiation of matrix crack (compare Figs 17b and 17c). Further explanations and findings are given in references 50 to 54 and 71.

Finite Element Analysis

In order to study the SIFs and displacements of the specimens actually tested, a FEM program with quarter-point singular elements was used. The quarter-point elements simulated the singular stress at the crack tip (72). The SIFs were calculated by means of a displacement-correlation technique. The path was approximated by series of 6.4 mm straight lines for both open and closed crack cases. In the open crack case a typical mesh for $\beta=36°$ is shown in Fig.18a. The initial mesh for a closed crack case and a typical mesh with kink extension of about 12.7 mm for $\beta=36°$ and the singular elements at the kink tip are shown in Figs. 18b and 18c. The details of the finite element analysis are described in references 50 to 53.

For both open and closed cracks, the FEM calculated value of the critical SIFs are shown in Fig.19. The values of K_I are not constant for different crack extensions, and they vary significantly for different values of β. When the calculated values of crack opening and sliding displacements using FEM are compared with the experimental data, a large discrepancy between the measured and the calculated values is seen (50 to 54,71). This discrepancy may be due to possible tractions that exist along matrix cracks (58,60,73,74). This observation motivated further investigation into the growth of matrix cracks. In mixed-mode loading conditions, both opening and sliding displacement occur and, as a result, both normal and shear tractions should be included in the analysis. The relation between shear and slip depends on crack opening, and sliding is accompanied by dilatancy (75,76). Ingraffea and Gerstle (77) have used in their finite element analysis of mixed mode crack propagation a shear stress versus slip relation suggested by Fenwick and Paulay (78). Several researchers have studied the relationship between shear stresses, normal stresses, and sliding and opening displacements (68,75,79).

To incorporate shear and normal tractions in the open crack case, for the $\beta=36°$ and $\beta=72°$ specimens, opening and sliding crack profiles were used. For two other inclinations ($\beta=18°$ and $\beta=54°$) and for all inclinations of closed crack, uniformly distributed traction forces were applied along the kink using clip gage measurements. The details of incorporating tractions along propagating crack are given in references 50,51 and 53.

By comparing Fig.19 and Fig.20, it can be seen that the values of $\bar{K}_{IC}$ (corrected critical values of K_I) now lie within a reasonable narrow band. In the closed crack case, the differences among K_{IC} values are still substantial (Fig.20). This may be due to the simplifying assumption that traction forces are uniformly distributed along the kink length. For both open and closed crack cases, the ratio of $\bar{K}_{IC}/\bar{K}_{II}$ is always numerically larger than one and it tends to increase with increasing kink extension (50,51,53). This shows the possibility of using a (K_I) max criterion to predict crack growth where the crack face tractions can be accurately known from previous experiments on their constitutive behavior. Hence it is also shown that a single parameter such as K_{IC} cannot describe crack propagation in concrete under mixed mode loading conditions. A similar conclusion has been reached above for mode I crack propagation by Jenq and Shah (46).

TOUGHENING OF CEMENT BASED MATERIALS DUE TO FIBER REINFORCING

The current state of practice in using fiber-reinforced concrete is limited to application of lower-fiber contents (about 1% by volume) (80,81).That is, fibers do not enhance the tensile-strain capacity of the cementitious matrix. However, recent research conducted jointly at Northwestern University and Denmark Technical University indicates that, as the volume fraction of fiber increases (up to 13%) and as fibers become more uniformly distributed, they can hinder growth of microcracks, suppress localization, and consequently, substantially increase fracture strength and strain capacity of the matrix (81 to 84).

The beneficial interaction between fiber and matrix is illustrated in Fig. 21. In this figure, the stress-strain curve of fiber-reinforced specimens subjected to uniaxial tension is shown.Specimens were reinforced with 12% and 8.7% volume of continuous polypropylene fibers by pulltrusion techniques developed at Denmark Technical

University.Also shown in Fig.21 are the calculated tensile stress-strain curves of the matrix.To obtain the stress-strain curve of the cement-based matrix, identical specimens were made (and tested under the same conditions)with fiber-reinforced epoxy and plain epoxy.The contribution of the fibers was calculated from these last two sets of tests.The contribution of the matrix was derived by subtracting the contribution of the fibers from the response of the composite. Fig.21 shows that the tensile strength of the cement based matrix can be as high as 16 MPa and that, even at a strain level of 1%, the matrix capacity can be more than 8 MPa.

Studies of fracture behavior of cement based materials reinforced with relatively high fiber volumes, conducted recently at Northwestern University, are briefly summarized below. Based on observed fracture mechanism an R-curve(fracture resistance curve)approach has been proposed to predict matrix toughening due to fiber reinforcement (81).

Propogation and Distribution of Cracks

Specimens-- Dog bone shaped specimens of dimensions 10x15x110 mm were cast in plexiglass molds and allowed to cure. The specimens were loaded in uniaxial tension.The loading was controlled by the average response of two LVDT's of 1.3 mm range mounted on the specimen.A 76 mm gage length was used in mounting of LVDT's on a knife edge assembly. A constant strain rate of $3x10^{-6}$ 1/s was used during testing.

Quantitative image analysis-- Thin sections of 20μm thickness were prepared and examined using both transmitted and reflected fluorescent light microscopy (Fig 22).The microscope was connected to a digital image analysis system for a quantitative fractography analysis.Using such automated techniques, important characteristics of microstucture in terms of true surface areas, lengths, orientation, size number, as well as distribution of these quantities were measured.

Laser holographic interferometry-- The experimental set-up for laser holographic interferometry is shown in Fig.23.Interferometric fringes can then be obtained by means of double exposure holography, where interference take place between the wave fronts reconstructed by two holograms of the same object at different stages of deformation recorded on the same plate. By analizing the fringes recorded, both crack length and opening displacement can be obtained.

Results and Discussions-- Crack propagation at different loading stages, obtained by using the laser holographic interferemetry, are shown in Fig.24. The toughening effect of fibers hinders unstable propagation of cracks, thus delaying localization.This phenomenon can be modelled by an R-curve approach.

R-Curve Approach

Based on fracture mechanism observed, an R-curve (fracture resistance curve) approach has been proposed by Shah and his colleagues to predict the toughening mechanism due to fiber reinforcement.A single-notch specimen subjected to unaxial tension is shown in Figure 25. R-Curve is defined as the locus of fracture energy release rates,G,for different size specimens with the same initial crack length (84,85).To account for the existence of stable crack growth, it was assumed that the critical crack length,a_c, is proportional to intial crack, a_o, (Fig.26):

$$a_c = a_o + \Delta a_c = \alpha a_o \quad \ldots\ldots\ldots\ldots \quad (11)$$

where Δa_c is precritical stable crack growth, and α_o is constant to be determined. G can be expanded into a Taylor's series, and a differential equation can be derived, which can be solved to obtained the R-curve. It should be noted that approximate critical conditions of $G_c = R_c$ and $a_c = \alpha a_o$ were used here. Considering only second order terms in the Taylor's series, the following R-curve formulation can be obtained:

$$R = \beta\ (a-a_o)^d \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12a)$$

$$d=1/2+(\alpha-1)/(\alpha-\{1/4+(\alpha-1)/a-[(\alpha-1)/\alpha]^2\}^{1/2} \quad \ldots\ldots \quad (12b)$$

where β is a constant to be determined. Eqs.(12) indicate that the R-curve is a function of the pre-critical growth. The material properties and the geometrical effects are included in the parameters α and β.

The values of α and β can be determined based on matrix properties: K^s_{Ic} and $CTOD_c$ introduced by Jenq and Shah (46) (described earlier under the two parameter fracture model) for specimens with different geometries. The critical crack length a_c is obtained by comparing the initial compliance with that obtained by unloading the specimen immediately after the peak load is reached in a CMOD controlled test. From these two compliances and the value of the maximum load, one can determine K^s_{Ic} and $CTOD_c$ using the relevant LEFM formula.

Because of the existence of fibers, stress intensity factor and crack tip opening displacement consist of contributions from far-field stresses and closing pressure. Therefore, crack stability conditions for the matrix are expressed as.

$$K^{s}_{IC} = K^{m}_{I} + K^{f}_{I} \quad \ldots\ldots\ldots\ldots\ldots \quad (13)$$

$$CTOD_{c} = CTOD_{m} + CTOD_{f} \quad \ldots\ldots\ldots \quad (14)$$

where K^{m}_{I} and $CTOD_{m}$ are the stress intensity and crack tip opening displacement at the tip of an effective traction free crack in the matrix due to far-field stress, and where K^{f}_{I} and $CTOD_{f}$ are the stress intensity and crack tip opening displacements due to the closing pressure. Once a_{c} is known, the values of α and β can be calculated from:

$$a_{c} = \alpha a_{o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (15a)$$

$$R^{s}_{IC} = (K^{s}_{IC})^{2}/E_{m} = (K^{m}_{IC}+K^{f}_{I}(a_{c}))^{2}/E_{m}$$

$$= \beta\ (\alpha a_{o} - a_{o})^{d} \quad \ldots\ldots \quad (15b)$$

where E_{m} is Young's modulus of the matrix.Details for the determination of α and β are given in references 84 and 85.

Theoretical an Experimental Comparisons

Response of unreinforced matrix-- The formulation discussed was applied to predict the response of unreinforced mortar subjected to uniaxial tension. The experimental results used were reported by Maji and Shah (58). The mortar used had a mix proportion of 1:2:0.5 of cement:sand water by weight. Specimens tested were 305x76x19 mm. A single edge notch of 12 mm was used, and the rate of opening of the notch tip was used as the control parameter in loading the specimens. Since required matrix properties for the mortar mix were available, the avarage values of material properties (K^{s}_{IC}, $CTOD_{c}$ and E_{m}) from two mortars, which have the mix proportions of 1:2.6:0.65 and 1:2.6:0.45 (cement:sand:water), respectively, and obtained from a three point bending specimen by Jenq and Shah (46), were used as an input for theoretical solutions. The R-curve obtained is shown in Fig.26. The details of the R-curve determination are given in references 81,85 and 86. It was assumed that the R value remains constant in the strain softening, post-critical regime. Fig.28 indicates a good agreement between the experimental and theoretical results.

Glass fiber composites-- Fig.29 represents the theoretical and experimental effects of fiber volume fraction on the bend over point (BOP) for glass fiber reinforced composites. The experimental results were obtained by Mobasher, Stang and Shah (83), Oakley and Proctor (87) and Ali, Majumdar and Singh (88). The theoretical results were obtained by using the material parameters (K^{s}_{IC}, and $CTOD_c$) for mortar as reported by Jenq and Shah (46) from the notched beam tests. Note that although varying different mixture proportions were used by different investigators, the theoretical curve corresponding to only one mortar is shown (81).

CONCLUDING REMARKS

In the light of the experimental observations and numerical analysis, using the fracture mechanics approach, outlined in this paper, following conclusions can be drawn:

1) For mode I case in both plain concrete and fiber-reinforced concrete, and for the combined mode I and mode II cases in plain concrete under compression, a single fracture parameter, such as the critical stress intensity factor cannot be simply applied to quasi-brittle cement-based materials. Thus, it is necessary to use nonlinear fracture mechanics models and their application should be extend beyond the linear elastic fracture mechanics.

2) To predict valid size-independent fracture parameters from small, laboratory-size specimens, Jenq and Shah have proposed a two-parameter fracture model. In this model, fracture process zone is simulated by an effective Griffith crack. The effective critical crack extension can be calculated from the two fracture parameters: critical stress intensity factor and critical crack tip opening displacement. The two parameter fracture model can also be extended to the tension-shear type of mixed mode loading condition. For very large structures such as dams, the relative dimensions of the nonlinear fracture process zone are small compared with the size of the structure and the linear elastic fracture mechanic can be considered valid.

3) The critical crack extension decreases with increasing compressive strength. This means that the higher the compressive strength, the more brittle the concrete is. Thus, the nonlinear fracture mechanics based model outlined here can provide a quantitative mean for determining the relative toughness of concrete.

4) In the combined mode I and mode II condition, under compresion, the corrected critical values of K_I at the tips of propagating cracks did not vary too much for different inclinations and crack extensions. The corrected K_I stress intensity factor at the tip of propagating crack was dominant for both open crack and stone-matrix interfacial crack cases, provided corrections were done by applying normal and shear tractions to match those observed experimentally at the various crack stages.

5) Based on fracture mechanism obtained from experimental observations, an R-curve approach can be used to predict the toughening of matrices due to fiber reinforcement. The matrix strength at the bend over point can be calculated for fiber reinforced composites. Although different mixture proportions were used by various researchers, the theoretical results show good agreement with experimental results.

ACKNOWLEDGEMENT

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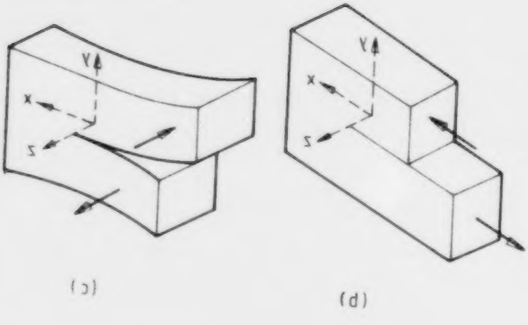


Fig.1 Loading modes of a crack

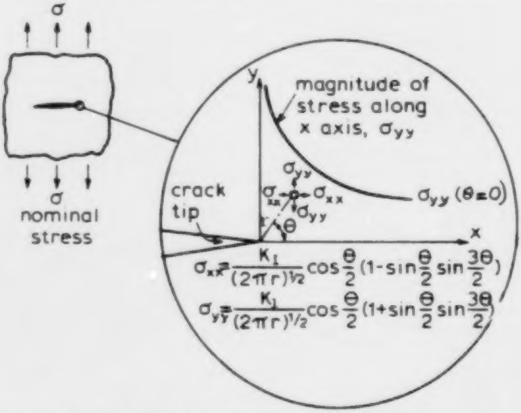

σ
σ
nominal
stress
crack
tip
y
magnitude of
stress along
x axis, σ_{yy}
σ_{yy}
σ_{xx}
σ_{xx}
σ_{yy}
$\sigma_{yy}\ (\theta=0)$
r
θ
x
$\sigma_{xx}=\frac{K_I}{(2\pi r)^{1/2}}\cos\frac{\theta}{2}(1-\sin\frac{\theta}{2}\sin\frac{3\theta}{2})$
$\sigma_{yy}=\frac{K_I}{(2\pi r)^{1/2}}\cos\frac{\theta}{2}(1+\sin\frac{\theta}{2}\sin\frac{3\theta}{2})$

Fig.2 Elastic-stress-field distribution ahead of a crack
From reference 36.

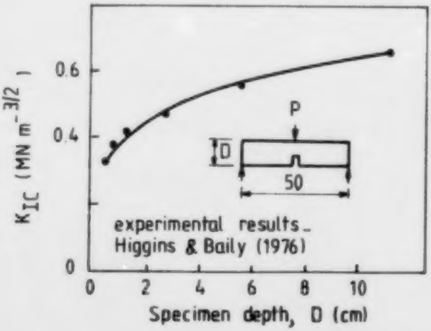


Fig.3 Size effect on conventional stress intensity factor
From reference 41.

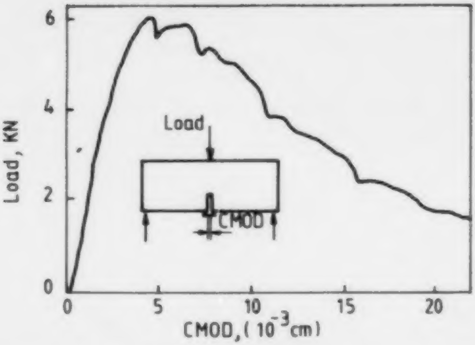


Fig.4 Load versus CMOD for plain mortar
From reference 42.

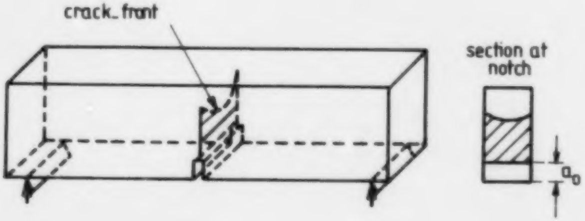


Fig.5 Variation of crack length along the width of specimen as observed using dye-penetration techniques
From reference 43.

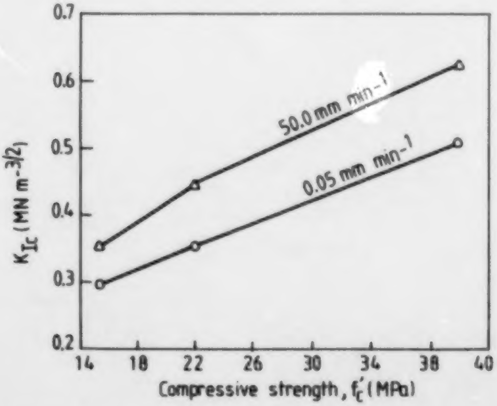


Fig.6 Effect of compressive strength and rate of loading on LEFM-based K_{Ic} of hardened cement paste
From reference 45.

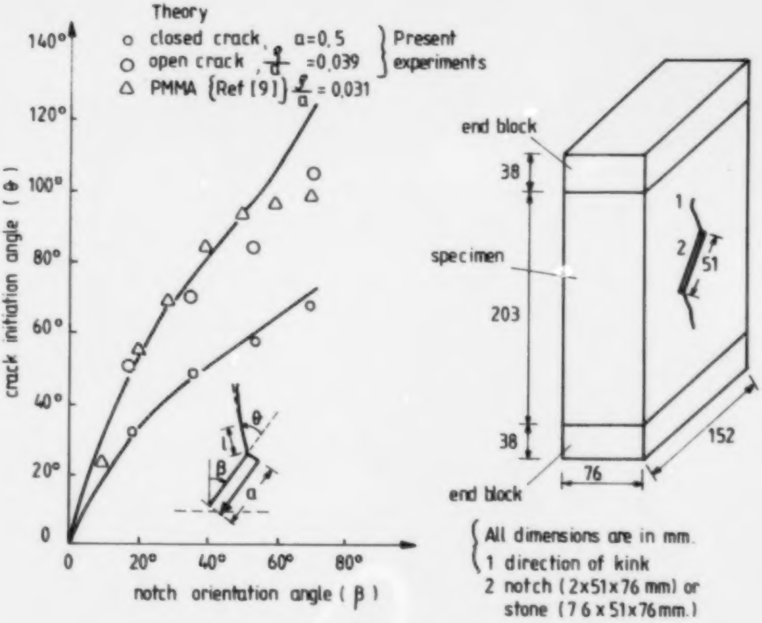


Fig.7 Comparison between theoretical and experimental crack initiation angles for both open and closed cracks
From reference 50.

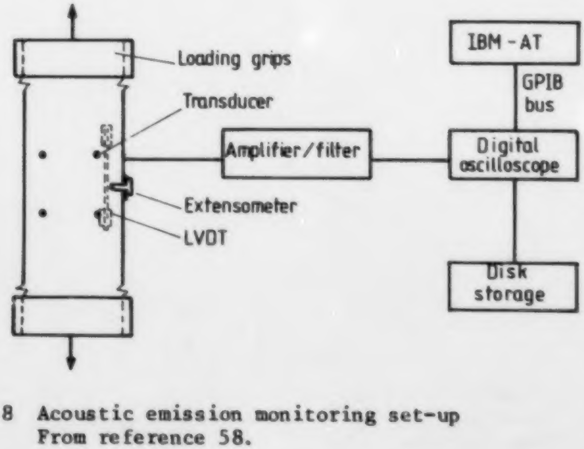

Loading grips
Transducer
Extensometer
LVDT
Amplifier/filter
IBM - AT
GPIB bus
Digital oscilloscope
Disk storage

Fig.8 Acoustic emission monitoring set-up
From reference 58.

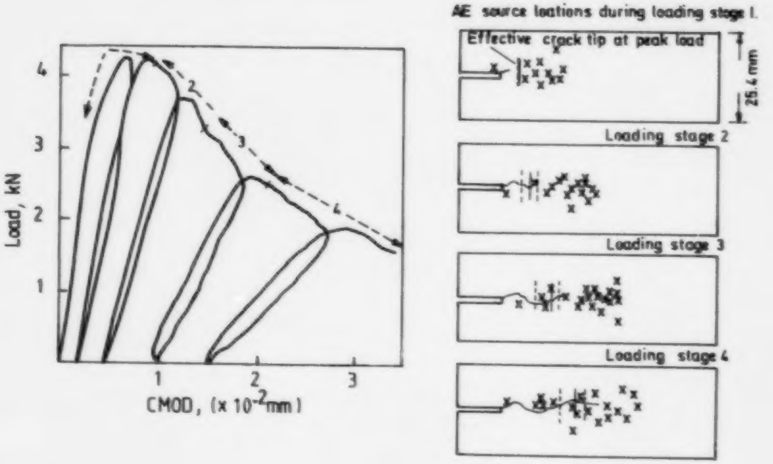


Fig.9 Microscopically observed crack lengths, acoustic emission source locations and effective crack lengths
From reference 58.

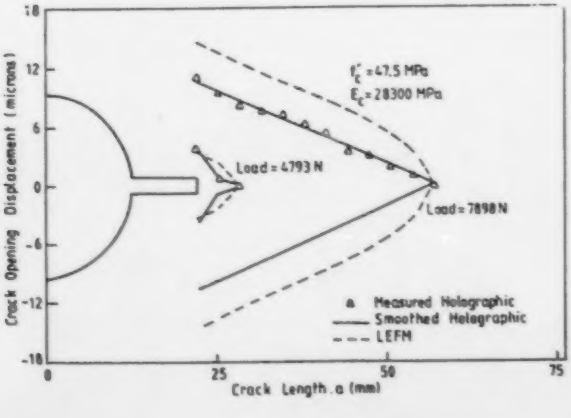


Fig.10 LEFM and holographic crack profiles
From reference 60.

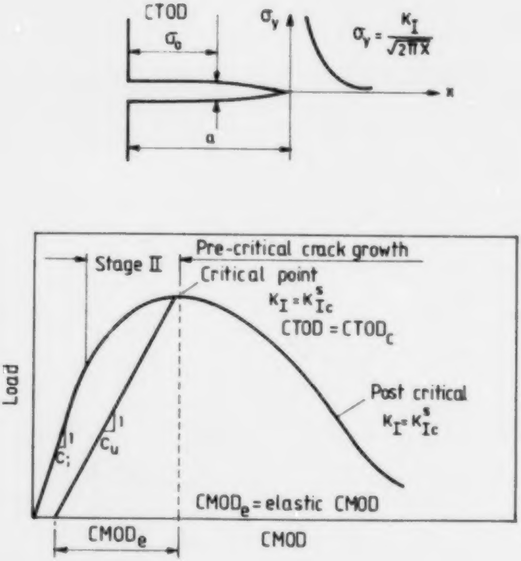

CTOD
σ_0
σ_y
$\sigma_y = \frac{K_I}{\sqrt{2\pi x}}$
x
a
Pre-critical crack growth
Stage II
Critical point
$K_I = K_{Ic}^s$
$CTOD = CTOD_c$
Load
Post critical
$K_I = K_{Ic}^s$
1
C_i
1
C_u
$CMOD_e$ = elastic CMOD
$CMOD_e$
CMOD

Fig.11 Explanation of terms used in the two parameter fracture model of concrete
From reference 46.

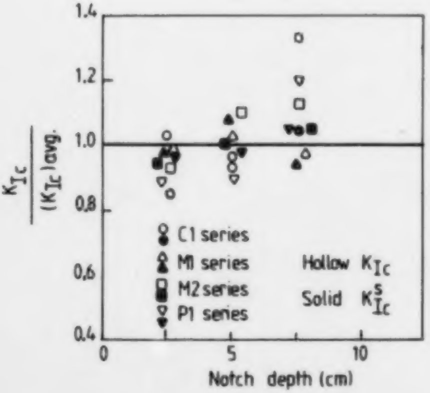


Fig.12 Size effect on K_{Ic}
From reference 38.

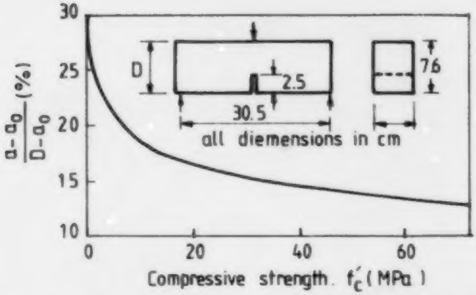


Fig.13 Decrease of nonlinearity (extent of prepeak crack growth) with increasing compressive strength From reference 70.

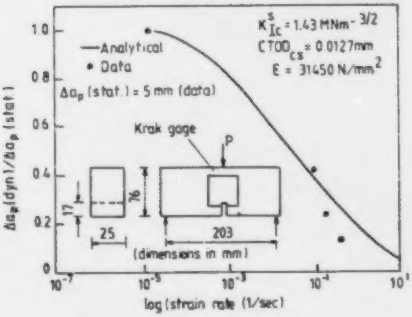


Fig.14 Decrease of nonlinearity (extent of prepeak crack growth) with increasing strain rate From reference 70.

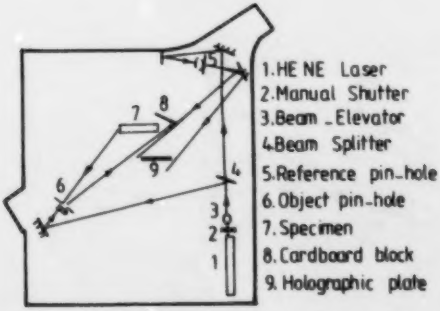

1
2
3
4
5
6
7
8
9
1.HE NE Laser
2.Manual Shutter
3.Beam _Elevator
4.Beam Splitter
5.Reference pin-hole
6.Object pin-hole
7.Specimen
8.Cardboard block
9.Holographic plate

Fig.15 Holographic interferometry set up From reference 53.

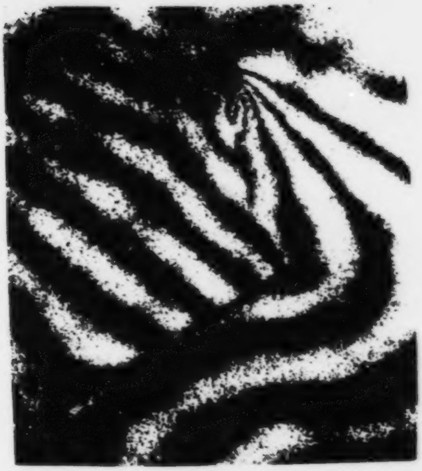
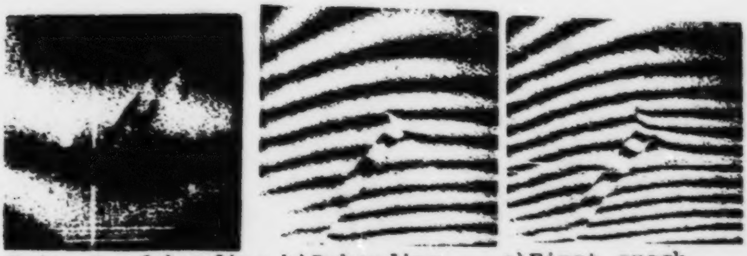
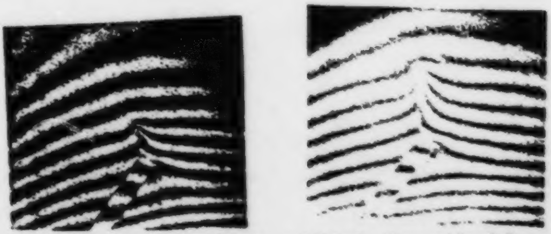


Fig.16. Holographic fringes for $\beta=54^{\circ}$ (open crack)
From reference 51.

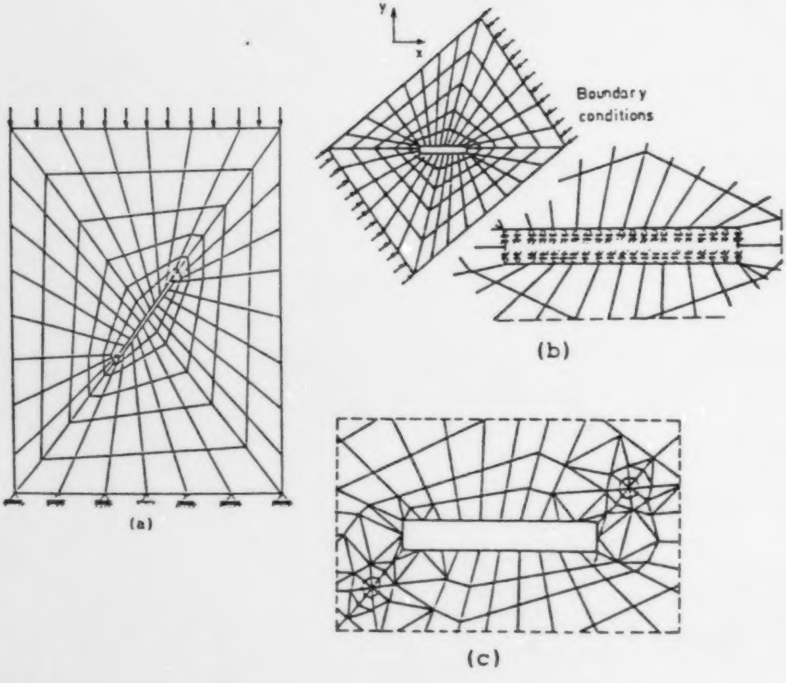


a)Prior to debonding b)Debonding c)First crack



d)Kink length=12mm e)Kink length=23mm

Fig.17 Holographic fringes for $\beta=36^{\circ}$ (stone-matrix interface crack)
From reference 53.


y
x
Boundary conditions

(a)

(b)

(c)

Fig.18 a)Initial FEM mesh with open notch for $\beta=36^{o}$
b)Initial FEM mesh with interfacial crack for $\beta=36^{o}$
c)FEM mesh showing propagated cracks and singular elements ($\beta=36^{o}$)
From reference 50.

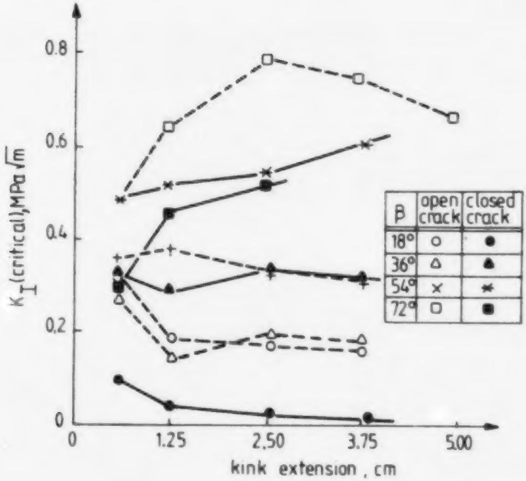


Fig.19 K_{IC} SIFs without tractions at the kink tips for both open and closed crack cases
From reference 50.

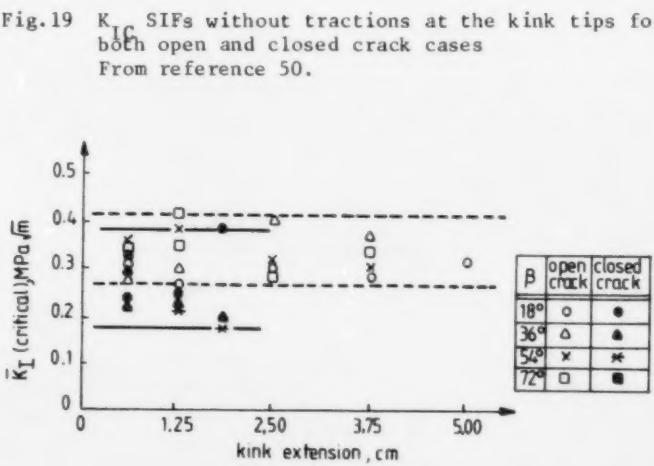


Fig.20 Corrected K_{IC} SIF versus kink extension for both open and closed crack cases
From reference 50.

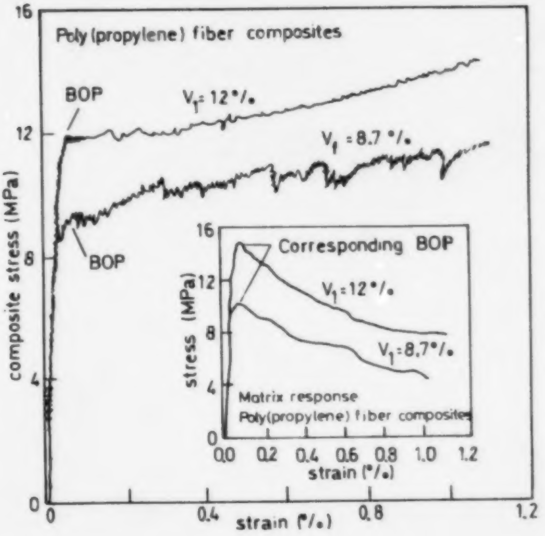


Fig.21 Stress-strain response of the composite material plotted for two-volume fraction of fibers. Computed response of the matrix phase is also shown in the inset
From reference 83.

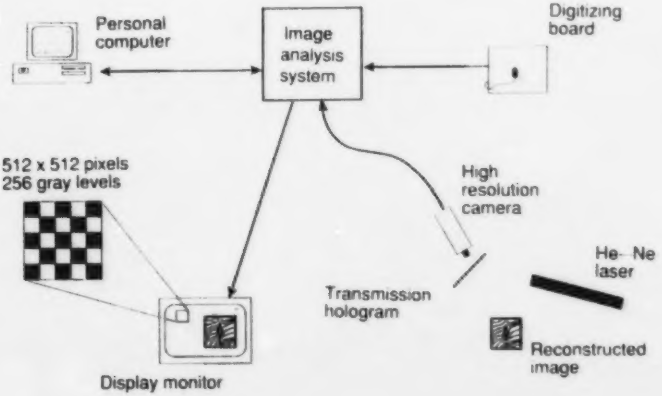

Personal computer
Image analysis system
Digitizing board
512 x 512 pixels
256 gray levels
High resolution camera
He- Ne laser
Transmission hologram
Reconstructed image
Display monitor

Fig.22 Image reconstruction and acquisition setup
From reference 84.

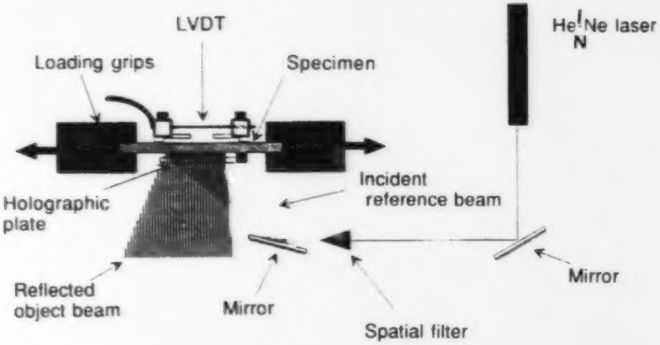
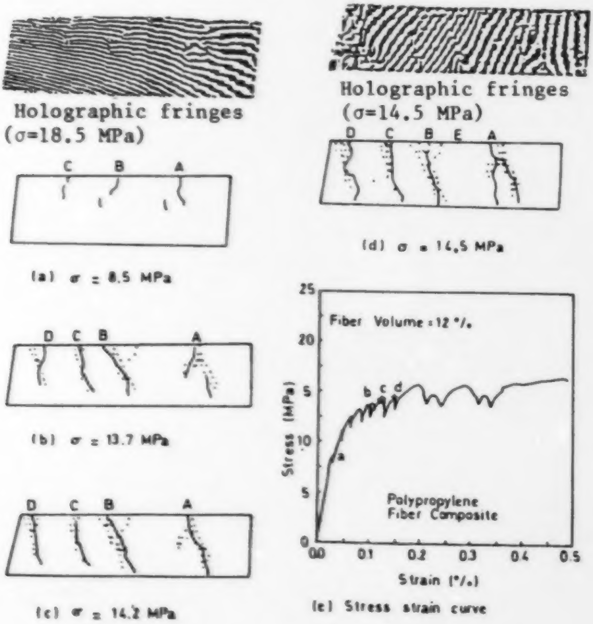

LVDT
He-Ne laser
Loading grips
Specimen
Incident reference beam
Holographic plate
Reflected object beam
Mirror
Spatial filter
Mirror

Fig.23 Optical arrangement of the holographic setup
From reference 84.


Holographic fringes (σ=18.5 MPa)

Holographic fringes (σ=14.5 MPa)

(a) σ = 8.5 MPa

(b) σ = 13.7 MPa

(c) σ = 14.2 MPa

(d) σ = 14.5 MPa

(e) Stress strain curve

Fig.24 Crack propagation at different loading stages of uniaxial tension specimen (e) stress-strain curve; (a) crack status at σ=8.5 MPa, (c) crack status at σ=14.2 MPa
From reference 86.

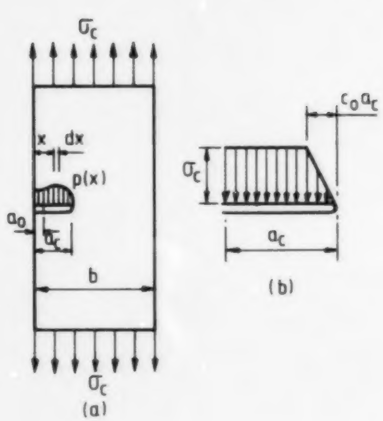

σ_c
x
dx
p(x)
a_0
a_c
b
σ_c
(a)
$c_0 a_c$
σ_c
a_c
(b)

Fig.25 Specimen geometry and closing pressure: (a) single notched specimen subjected to tension; (b) simplified bilinear closing pressure
From reference 86.

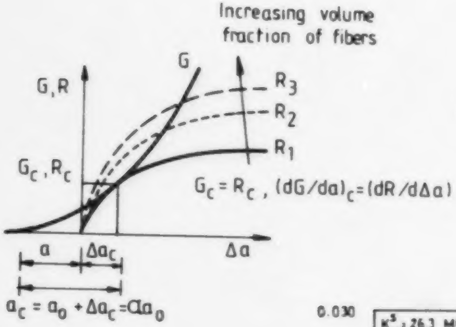

Increasing volume fraction of fibers
G
G, R
R_3
R_2
R_1
G_c, R_c
$G_c = R_c$, $(dG/da)_c = (dR/d\Delta a)$
a
Δa_c
Δa
$a_c = a_0 + \Delta a_c = \alpha a_0$

Fig.26 Schematics of R-curve for matrix's response as affected by fibers
From reference 86.

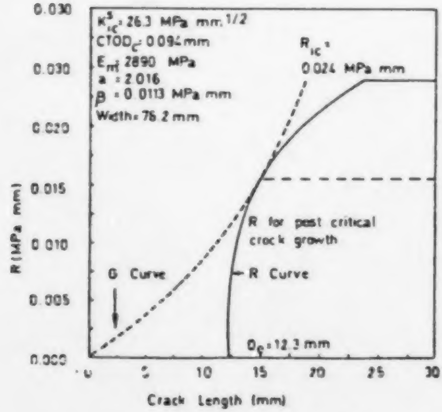


Fig.27 R and G curves for unreinforced cement-based matrix
From reference 86.

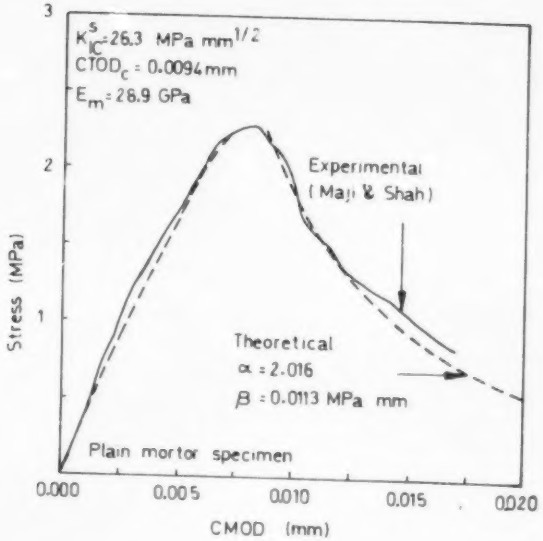


Fig.28 Stress-CMOD curve for unreinforced cement-based matrix
From reference 86.

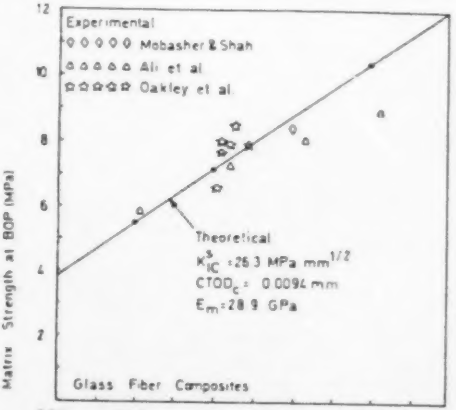


Fig.29 Matrix strengths for glass fiber composites
From reference 81.

Cracking of Concrete:
ACI and CEB Approaches

by Edward G. Nawy

Synopsis: This paper covers a review of the state of the art on most aspects of cracking and an overall evaluation of the flexural crack width development and crack control in concrete structures. It is based on extensive research over the past five decades in the United States and overseas in the area of cracking in reinforced and prestressed concrete beams and two-way action slabs and plates including the mechanics of micro-cracking. It also presents a concensus interpretation of the major causes of plastic and drying shrinkage cracking and crack development in mass concrete.

Control of cracking has become essential in order to maintain the integrity and aesthetics of concrete structures. The trend is stronger than ever in better utilization of concrete strength, use of higher strength concretes including super-strength concretes of 20,000 psi (138 MPa) compressive strength and higher, more prestressed concretes and increased uses of limit failure theories. Hence, a closer control is needed of serviceability requirements in cracking and deflection behavior.

Commonly accepted procedures and expressions are presented for the control of cracking in reinforced concrete beams and thick one-way slabs, prestressed pretensioned and post-tensioned flanged beams and reinforced concrete two-way action structural floor slabs and plates. They are mainly based on the Codes and Standard Reports of the ACI and the CEB in their latest versions. Modifying reduction factors for crack width evaluation are also given for beams having concrete strength in excess of 6,000 psi (41.4 MPa). In addition, recommendations are given for the maximum tolerable flexural crack widths in concrete elements, limitation of shrinkage cracking and control of cracking in mass concrete.

Keywords: **Beams, concrete, compressive strength, cover, cracks, crack control, crack width, deflection, equations, flexural stress, fracture coefficients, fracture mechanics, macro-cracking, mass concrete, micro-cracking, plastic, plates, post-tensioned, prestressed, pretensioned, reinforcement, shrinkage, stabilized spacing, tanks, temperature, two-way action, volumetric change.**

Edward G. Nawy, FACI, is Professor of Civil Engineering, Rutgers, The State University of New Jersey, holding the distinguished professor rank (Professor II). Active in the ACI since 1949, Prof. Nawy is a former chairman and current member of ACI Committee 224, Cracking, and is currently a member of ACI Committee 340, Strength Design Aids for ACI Building Codes, Chairman of ACI Committee 435, Deflection of Concrete Building Structures; member of Joint ACI-ASCE Committee 421, Design of Reinforced Concrete Slabs; the Concrete Materials Research Council; Technical Activities committee of PCI, and the Tall Buildings Council. His research interests are in reinforced and prestressed concrete, particularly in the areas of crack control and serviceability behavior. Dr. Nawy has published more than 115 papers and is the author of three major textbooks: Reinforced Concrete - A Fundamental Approach, (translated into Spanish and Chinese), Simplified Reinforced Concrete, and Prestressed Concrete - A Fundamental Approach as well as numerous handbook chapters. He holds many honors and awards, including the ACI Chapter Activities Committee and Henry L Kennedy Awards, was twice President of the N.J. ACI chapter, served two terms on the Rutgers University Board of Governors, was Honorary Vice President of the RILEM 30th Anniversary Congress, and is Honorary Professor of NIT, the Nanjing Insitute of Technology. He is consultant to major engineering organizations in the New York-New Jersey metropolitan area, and is a registered Professional Engineer in New Jersey, New York, Pennsylvania and California, and chartered engineer in England and the Commonwealth.

INTRODUCTION

Presently, the trend is stronger than ever in better utilization of concrete strength, use of super higher strength concretes of 20,000 psi (138 MPa) compressive strength and higher, use of high strength reinforcement, more prestressed concretes and increased use of limit failure theories - all these trends require closer control of serviceability requirements in cracking and deflection behavior. Hence, knowledge of the cracking behavior of concrete elements becomes essential.

Concrete cracks early in its loading history. Most cracks are a result of the following actions to which concrete can be subjected:

1. Volumetric change caused by plastic shrinkage drying shrinkage, creep under sustained load, thermal stresses including elevated temperatures, and chemical incompatibility of concrete components including the presence of alkali reactive aggregates.

2. Direct stress due to applied loads or reactions or internal stress due to continuity, reversible fatigue load, long-term deflection, camber in prestressed systems, or environmental effects including differential movement in structural systems.

3. Flexural stress due to bending moments caused by transverse loads.

While the net result of these three actions is the formation of cracks, the mechanisms of their development cannot be considered identical. Volumetric change generates internal microcracking which may develop into full cracking, while direct internal or external stress or applied loads and reactions could either generate internal microcracking, such as in the case of fatigue due to reversible load, or flexural macrocracking leading to fully developed cracking.

This paper is intended to be a world review of the state-of-the-art on cracking in concrete. While the coverage cannot be all-inclusive, yet, an attempt is made to include within the limited space available most of the current information that is widely accepted on this subject.

MICRO-CRACKING

Micro-cracking can be mainly classified into two categories: a) bond cracks at the aggregate-mortar interface, b) paste cracks within the mortar matrix. Interfacial bond cracks are caused by interfacial shear and tensile stresses due to early volumetric change without the presence of external load. Volume change caused by hydration and shrinkage could create tensile and bond stresses of sufficient magnitude as to cause failure at the aggregate-mortar interface (1,2). As the external load is applied, mortar cracks develop due to increase in compressive stress, propagating continuously through the cement matrix up to failure. A typical schematic stress-strain diagram (Fig. 1) shows the nonlinear relationship developed early in the stress history and starting with bond micro-cracking. While extensive work exists in the area of volumetric change cracking, the need is apparent for additional work on creep effects on micro-cracking and also for the development of a universally acceptable fracture theory to interrelate the nonlinear behavioral factors resulting in crack propagation.

It appears that the damage to cement paste seems to play a significant role in controlling the stress-strain relationship in concrete. The coarse aggregate particles act as stress-raisers that decrease the strength of the cement paste. As a result micro-cracks develop that can only be detected by large magnification. The importance of additional work lies not only in the evaluation of the micro-cracks, but also in their significance on the development of macro-cracks which generate from those micro-cracked centers of plasticity.

Application of the fracture mechanics principles to the micro-cracking in concrete has thus become necessary. Several theories have been developed (3,4,5). In general, investigators have proposed the use of the effective crack length to account for the fracture process zone and deduced from the reduction in stiffness at the peak load in a 3-point bend test. In these investigations, the specimen size effect and the material properties were considered to be the dominant factors on the fracture in concrete

Cracking Due to Shrinkage

Basically, there are two types of shrinkage: plastic shrinkage and drying shrinkage. *Plastic shrinkage* occurs during the first few hours after placing fresh concrete in the forms. Exposed surfaces such as floor slabs are more easily affected by exposure to dry air because of their large contact surface. In such cases, moisture evaporates faster from the concrete surface than it is replaced by the bleed water from the lower layers of the concrete elements (2). *Drying shrinkage*, on the other hand, occurs after the concrete has already attained its final set and a good portion of the chemical hydration process in the cement gel has been accomplished.

Drying shrinkage is the decrease in the volume of a concrete element when it loses moisture by evaporation. The opposite phenomenon, that is, volume increase through water absorption, is termed *swelling*. In other words, shrinkage and swelling represent water movement out of or into the gel structure of a concrete specimen due to the difference in humidity or saturation levels between the specimen and the surroundings irrespective of the external load.

Shrinkage is not a completely reversible process. If a concrete unit is saturated with water after having fully shrunk, it will not expand to its original volume. Figure 2 relates the increase in shrinkage strain ε_{sh} with time. The rate decreases with time since older concretes are more resistant to stress and consequently undergo less shrinkage, such that the shrinkage strain becomes almost asymptotic with time.

Several factors affect the magnitude of drying shrinkage:

1. *Aggregate*. The aggregate acts to restrain the shrinkage of the cement paste; hence concretes with high aggregate content are less vulnerable to shrinkage. In addition, the degree of restraint of a given concrete is determined by the properties of aggregates; those with high modulus of elasticity or with rough surfaces are more resistant to the shrinkage process.

2. *Water/cement ratio*. The higher the water/cement ratio, the higher the shrinkage effects. Figure 3 is a typical plot relating aggregate content to water/cement ratio.

3. *Size of the concrete element*. Both the rate and total magnitude of shrinkage decrease with an increase in the volume of the concrete element. However, the duration of shrinkage is longer for larger members since more time is needed for drying to reach the internal regions. It is possible that 1 year may be needed for the drying process to begin at a depth of 10 in. from the exposed surface, and 10 years to begin at 24 in. below the external surface.

4. *Medium ambient conditions*. The relative humidity of the medium affects greatly the magnitude of shrinkage; the rate of shrinkage is lower at high states of relative humidity. The environment temperature is another factor, in that shrinkage becomes stabilized at low temperatures.

5. *Amount of reinforcement*. Reinforced concrete shrinks less than plain concrete; the relative difference is a function of the reinforcement percentage.

6. *Admixtures*. This effect varies depending on the type of admixture. An accelerator such as calcium chloride, used to accelerate the hardening and setting of the concrete, increases the shrinkage. Pozzolans can also increase the drying shrinkage, whereas air-entraining agents have little effect.

7. *Type of cement*. Rapid-hardening cement shrinks somewhat more than other types, while shrinkage-compensating cements minimize or eliminate shrinkage cracking if used with restraining reinforcement.

8. *Carbonation*. Carbonation shrinkage is caused by the reaction between carbon dioxide (CO_2) present in the atmosphere and that rpresent in the cement paste. The amount of the combined shrinkage varies according to the sequence of occurrence of carbonation and drying processes. If both phenomena take place simultaneously, less shrinkage develops. The process of carbonation, however, is dramatically reduced at relative humidities below 50%.

In order to control shrinkage cracking by minimizing its extent, reduction of cracking tendency can be achieved by addressing all remedial measures that take into account the eight major factors presented.

FLEXURAL CRACKING

External load results in direct and bending stresses causing flexural, bond and diagonal tension cracks. Immediately after the tensile stress in the concrete exceeds its tensile strength at a particular location, internal micro-cracks form. These cracks generate into macro-cracks propagating to the external fiber zones of the element.

Immediately after the full development of the first crack in a reinforced concrete element, the stress in the concrete at the cracking zone is reduced to zero and is assumed by the reinforcement (2). The distribution of ultimate bond stress, longitudinal tensile stress in the concrete and longitudinal tensile stress in the steel can be schematically represented in Fig. 4.

Crack width is a primary function of the deformation of reinforcement between the two adjacent cracks 1 and 2 in Fig. 4, if the small concrete strain along the crack interval a_c is neglected. The crack width would hence be a function of the crack spacing to the load level at which no more cracks develop, leading to the stabilization of the crack spacing (Fig. 5)..

The major parameters affecting the development and characteristics of the cracks are: percentage of reinforcement, bond characteristics and size of bar, concrete cover, and the concrete stretched area, namely, the concrete area in tension. On this basis, one can propose the following mathematical model:

$$w = \alpha \, a_c^{\beta'} \, \epsilon_s^{\gamma} \qquad (1)$$

where w = maximum crack width, and
α, β' and γ are nonlinearity constants. Crack spacing a_c is a function of the factors enumerated previously, being inversely proportional to bond strength and active steel ratio (steel percentage in terms of the concrete area in tension). ϵ_s is the strain in the reinforcement induced by external load.

The basic mathematical modal in equation (1) with the appropriate experimental values of the constants α, β and γ can be derived for the particular type of structural member. Such a member can be a one-dimensional element such as a beam, a two-dimensional structure such as a two-way slab, or a three-dimensional member such as a shell or circular tank wall. Hence, it is expected that different forms or expressions apply for the evaluation of the macro-cracking behavior of different structural elements consistent with their fundamental structural behavior (Ref. 2-10).

FLEXURAL CRACKING AND CRACK CONTROL IN REINFORCED CONCRETE BEAMS AND THICK ONE-WAY SLABS

Requirements for crack control in beams and thick one-way slabs (10 in. or thicker) in the ACI Building Code (ACI 318) are based on the statistical analysis of maximum crack width data from a number of sources. Based on the analysis, the following general conclusions were reached (6-9):

1. The steel stress is the most important variable.
2. The thickness of the concrete cover is an important variable, but not the only geometric consideration..
3. The area of concrete surrounding each reinforcing bar is also an important geometric variable.
4. The bar diameter is not a major variable.
5. The size of the bottom crack width is influenced by the amount of strain gradient from the level of the steel to the tension face of the beam.

The simplified expression relating crack width to steel stress is given in Eq. 2 as follows (Ref. 6).

$$w_{max} = 0.076\,\beta\, f_s \sqrt[3]{d_c A} \times 10^{-3} \qquad (2)$$

where f_s = reinforcing steel stress, ksi

A = area of concrete symmetric with reinforcing steel divided by number of bars, in.^{2}

d_c = thickness of concrete cover measured from extreme tension fiber to center of bar or wire closest thereto, in.

β = h_2/h_1 where h_1 = distance from neutral axis to the reinforcing steel, in.

and h_2 = distance from neutral axis to extreme concrete tensile surface.

A plot relating the reinforcement strength to the ratio of the concrete area in tension to the reinforcement area is shown in Fig. 6 for all bar sizes.

In the ACI Code, when the design yield strength f_y for tension reinforcement exceeds 40 ksi, cross sections of maximum positive and negative moment have to be so proportioned that the quantity z given by Eq. 3.

$$z = f_s \sqrt[3]{d_c A} \qquad (3)$$

does not exceed 175 kips per in. for interior exposure and 145 kips per in. for exterior exposure. Calculated stress in the reinforcement at service load f_s(ksi) shall be computed as the moment divided by the product of steel area and internal moment are. In lieu of such computations, it is permitted to take f_s as 60 percent of specified strength f_y.

When the strain, ϵ_s, in the steel reinforcement is used instead of stress, f_s, Eq. (3) becomes

$$w = 2.2 \beta \epsilon_s \sqrt[3]{d_c A} \tag{4}$$

Eq. (4) is valid in any system of measurement.

The cracking behavior in thick one-way slabs is similar to that in shallow beams. For one-way slabs having a clear concrete cover in the range of 1 in. (25.4 mm), (Eq. 4) can be adequately applied if β = 1.25 to 1.35 is used.

CEB Recomendations

Crack control recommendations proposed the European Model Code for Concrete Structures are supposed to apply to both prestressed and reinforced concrete and can be summarized as follows (11):

The mean crack width, w_m, in beams is expressed in terms of the mean crack spacing, s_{rm}, such that

$$w_m = \epsilon_{sm} s_{rm} \tag{5}$$

where

$$\epsilon_{sm} = \frac{f_s}{E_s} [1 - \chi (\frac{f_{sr}}{f_s})^2] \leq 0.4 \frac{f_s}{E_s} \tag{6}$$

and represents the average strain in the steel.

f_s = steel stress at the crack

f_{sr} = steel stress at the crack due to forces causing cracking at the tensile strength of concrete

χ = bond coefficient, 1.0 for ribbed bars, reflecting influence of load repetitions and load duration

The mean crack spacing is

$$s_{rm} = 2(c - \frac{s}{10}) + \chi_2\chi_3 \frac{d_b}{Q_R} \tag{7}$$

where

c = clear concrete cover

s = bar spacing, limited to $15d_b$

χ_2 = 0.4 for ribbed bars

χ_3 = depends on the shape of the stress diagram, 0.125 for bending

Q_R = A_s/A_t

A_t = effective area in tension, depending on arrangement of bars and type of external forces; it is limited by a line $c + 7d_b$ from the tension face for beams; in the case of thick slabs, not more than halfway to the neutral axis

A simplified formula can be derived for the mean crack width in beams with ribbed bars.

$$w_m = 0.7 \frac{f_s}{E_s} (3c + 0.05 \frac{d_b}{Q_R}) \tag{8}$$

A characteristic value of the crack width, presumably equivalent to the probable maximum value, is given as $0.7w_m$.

The 1991 CEB model code changes on crack width evaluations have modified Eqs. 5,6, and 7 to Eqs. 5a and 7a as follows

$$w_k = \ell_{s,max} (\epsilon_{sm} - \epsilon_{cm} - \epsilon_{cs}) \tag{5a}$$

where

ϵ_{sm} = average steel strain over length $\ell_{s,max}$

ϵ_{cm} = Average concrete strain over length $\ell_{x,max}$

ϵ_{cs} = strain of concrete due to shrinkage

and

$$\ell_{s,max} = 2(s_o + \frac{\sigma_{s2} - \sigma_{sE}}{4\tau_{bk}} \phi_s) \tag{7a}$$

s_o = length at crack vicinity along which bond stresses have negligiable values

σ_{s2} = steel stress in crack

σ_{sE} = steel stress at point of zero slip

$\tau = 2f'_t$ for deformed bars

$b_k = 1\ f'_t$ for plain bars

ϕ_s = steel bar diameter

The British Code of Practice has essentially a similar approach to the CEB recommendations presented in this section. It differentiates, however, in the application of the expressions to reinforced as compared to prestressed concrete beams.

It should be emphasized that all these expressions were developed on the basis of beam tests where the concrete compressive strength was in the range of 4,000-5,000 psi (27.6-34.5 MPa). For beams whose concrete strength exceeds 6,000 psi (41.4 MPa), the predicted crack width should be proportionally reduced but not necessarily linearly. More research is needed to establish the reduction factors applicable.

The Australian Code on Flexural Crack Control

The Australian Code does not recommend any formula for the calculation of crack widths. Crack control for flexure in reinforced concrete beams is achieved if the center-to-center spacing of bars near the tension face of the beam does not exceed 8 in. (200 mm) and the distance from the side or soffit to the center of the nearest longitudinal bar is not greater than 4 in. (100 mm). In the case of fully prestressed concrete beams, the maximum tensile stress in the concrete due to short-term service loads should not exceed $3\sqrt{f'_c}$. To control flexural cracking in partially prestressed concrete beams, the increment in steel stress near the tension face is limited to 29 Ksi (200 MPa), as the load increases from its value when the extreme concrete tensile fiber is at zero stress to the short-term service load value; and the center-to-center spacing of reinforcement, including bonded tendons, is limited to 8 in. (200 mm)

Flexural cracking in reinforced concrete slabs is controlled by limiting the center-to-center spacing of bars in each direction to the lesser of 2.5 times the thickness of slab or 20 in. (500 mm). In fully prestressed slabs, similar to beams, the maximum tensile stress in the concrete due to short-term service loads is limited to $3\sqrt{f'_c}$. For partially prestressed slabs, the incremental steel stress should not exceed 22 Ksi (150 MPa) and the center-to-center spacing of reinforcement including bonded tendons, is not to exceed 20 in. (500 mm).

It should be noted that the extensive Nawy work (Ref. 7,8) demonstrated that the maximum crack spacing in two-way reinforced concrete slabs should not exceed 12 in. (300 mm), otherwise yield line wide cracks would be prematurely generated. Hence, the ACI 318 Code limits the maximum spacing to twice the slab thickness.

FLEXURAL CRACKING AND CRACK CONTROL IN PRESTRESSED PRETENSIONED AND POST-TENSIONED BEAMS

The increased use of partial prestressing, allowing limited tensile stresses in the concrete under service and overload conditions while allowing nonprestressed steel to carry the tensile stresses, is becoming prevalent due to practicality and economy. Consequently, an evaluation of the flexural crack widths and spacing and control of their development become essential. Work in this area is relatively limited because of the complications induced by the various factors affecting crack width development in prestressed concrete and the high cost of testing for these parameters. However, experimental investigations support the hypothesis that the major controlling parameter is the reinforcement stress change beyond the decompression stage. Nawy, et al, have undertaken extensive research since the 1960s on the cracking behavior of prestressed pretensioned and post-tensioned beams and slabs because of the great vulnerability of the highly stressed prestressing steel to corrosion and other environmental effects and the resulting premature loss of prestress (Refs. 13, 14). Other investigators (Ref. 15,16) have extended this work under varying load combinationns. From all the available studies, it is evident that serviceability behavior under service and overload conditions can be controlled by the design engineer through the application of the criteria presented in this section.

Mathematical Model Formulation for Serviceability Evaluation

Crack spacing -- Primary cracks form in the region of maximum bending moment when the external load reaches the cracking load. As loading is increased, additional cracks will form and the number of cracks will be stabilized when the stress in the concrete no longer exceeds its tensile strength at further locations regardless of load increase. This condition is important as it essentially produces the absolute minimum crack spacing which can occur at high steel stresses, to be termed the stabilized minimum crack spacing. The maximum possible crack spacing under this stabilized condition is twice the minimum, to be termed the stabilized maximum crack spacing. Hence, the stabilized mean crack spacing a_{cs} is deduced as the mean value of the two extremes.

The total tensile force T transferred from the steel to the concrete over the stabilized mean crack spacing can be defined as

$$T = \gamma \, a_{cs} \, \mu \, \Sigma o \tag{9a}$$

where

γ = a factor reflecting the distribution of bond stress

μ = maximum bond stress which is a function of $\sqrt{f'_c}$

Σo = sum of reinforcing elements' circumferences

Fig. 7 illustrates the forces that cause the formulation of the stabilized crack.

The resistance R of the concrete area in tension A_t can be defined as

$$R = A_t \, f'_t \tag{9b}$$

where f'_t = tensile splitting strength of the concrete. By equating Eqs. 9a and 9b, the following expression for a_{cs} is obtained, where c is a constant to be developed from the tests:

$$a_{cs} = c \, \frac{A_t \, f'_t}{\Sigma o \, \sqrt{f'_c}} \tag{10a}$$

The concrete stretched area, namely the concrete area A_t in tension for both the evenly distributed and non-evenly distributed reinforcing elements, is illustrated in Fig. 8. With a mean value of $f'_t / \sqrt{f'_c} = 7.95$ in this investigation (13), a regresson analysis of the test data resulted in the following expression for the mean stabilized crack spacing

$$a_{cs} = 1.20 \, A_t / \Sigma o \tag{10b}$$

Crack width -- If Δf_s is the net stress in the prestressed tendon or the magnitude of the tensile stress in the normal steel at any crack width load level in which the decompression load (decompression here means $f_c = 0$ at the level of the reinforcing steel) is taken as the reference point (Ref. 11-14), then for the prestressed tendon (Refs. 11-14),

$$\Delta f_s = f_{nt} - f_d \tag{11}$$

where

f_{nt} = stress in the prestressing steel at any load beyong the decompression load

f_d = stress in the prestressing steel corresponding to the decompression load

The unit strain $\epsilon_s = \Delta f_s/E_s$. It is logical to disregard as insignificant the unit strains in the concrete due to the effects of temperature, shrinkage and elastic shortening. The maximum crack width as defined in Eq. 1 can be taken as

$$w_{max} = k\ a_{cs}\ \epsilon_s^{\alpha} \tag{12a}$$

where k and α are constants to be established by tests, or

$$w_{max} = k'\ a_{cs}(\Delta f_s)^{\alpha} \tag{12b}$$

where k' is a constant in terms of constant k.

Expressions for Pretensioned Beams

Eq. 12a is rewritten in terms of Δf_s so that analysis of the test data of all the simply supported test beams in this work leads to the following expression at the reinforcement level:

$$w_{max} = 1.4 \times 10^{-5}\ a_{cs}\ (\Delta f_s)^{1.31} \tag{13}$$

Linearizing Eq. 13 for easier use by the design engineer leads to the following simplified expression of the maximum crack width at the reinforcing steel level:

$$w_{max} = 5.85 \times 10^{-5}\ \frac{A_t}{\Sigma o}\ (\Delta f_s) \tag{14a}$$

and a maximum crack width (in.) at the tensile face of the concrete:

$$w'_{max} = 5.85 \times 10^{-5}\ R_i\ \frac{A_t}{\Sigma o}\ (\Delta f_s) \tag{14b}$$

where R_i is the distance ratio $= h_2/h_1$ with h_2 being the distance from the neutral axis to the extreme tension fibers and h_1 the distance from the neutral axis to the reinforcement centroid.

A plot of the pretensioned beams tests data and the best fit expression for Eq. 14a is given in Fig. 9 with a 40 percent spread, which is reasonable in view of the randomness of crack development and the linearization of the original expression of Eq. 13.

Expressions for Post-Tensioned Beams

The expression developed for the crack width in post-tensioned bonded beams which contain mild steel reinforcement is

$$w_{max} = 6.51 \times 10^{-5} \frac{A_t}{\Sigma o} (\Delta f_s) \qquad (15a)$$

for the width at the reinforcement level closest to the tensile face, and

$$w_{max} = 6.51 \times 10^{-5} R_1 \frac{A_t}{\Sigma o} (\Delta f_s) \qquad (15b)$$

at the tensile face of the concrete lower fibers.

For nonbonded beams, the factor 6.51 in Eqs. 15a and 15b becomes 6.83. A plot of the data and the best fit expression for Eq. 15a is given in Fig. 10.

A typical plot of the effect of the various steel percentages on the crack spacing at the various stress levels Δf_s is given in Fig. 11. It is seen from this plot that crack spacing stabilizes at a net stress level range of 30 to 36 ksi.

Cracking of High Strength Prestressed Beams

Recent work was completed at Rutgers University (1991) on the cracking behavior of pretensioned and nonbonded post-tensioned beams having cylinder compressive strengths in the range of 10,200 psi to 12,300 psi (70.3 to 84.8 MPa). Analysis of the test data has led to introducing modifying reduction multipliers in order to include the effect of the high concrete compressive and tensile strength on the stabilized crack width in prestressed beams.

A multiplier λ_r should be used in Eqs. 14a and 14b for pretensioned beams and λ_o should be used in Eqs. 15a and 15b for nonbonded post-tensioned beams.

$$\lambda_r = \frac{2}{(0.75+0.06\sqrt{f'_c})\sqrt{f'_c}} \qquad (16a)$$

and

$$\lambda_o = \frac{1}{0.75+0.06\sqrt{f'_c}} \qquad (16b)$$

where f'_c is in ksi units.

Other Work on Cracking in Prestressed Concrete

Based on the analysis of results of various investigators, Naaman (17) produced the following modified expression for partially prestressed pretensioned members

$$w_{max} = [42 + 5.58 \frac{A_t}{\Sigma o} (\Delta f_s)] \times 10^{-5} \qquad (17)$$

This regression expression is very close to Eq. 14 by the author. If plotted against the experimental results of the various researchers it gives a best fit as shown in Fig. 12.

A comparison is made between the author's equations and the basic CEB-FIP approach using similar notations.

Nawy: $w_{max} = \xi (\frac{A_t}{\Sigma o}) \Delta\sigma_p$

CEB-FIP: $w_{max} = [K\frac{\phi}{\rho_t} + \alpha_o] \frac{\Delta\sigma_p}{E_s} [1 - \beta(\frac{\Delta\sigma_{pr}}{\Delta\sigma_p})^2]$

where

$1/\Sigma o = \lambda(\frac{\phi}{\rho_t})$ using λ as a multiplier in lieu of K & β which are test parameters, and ϕ is the reinforcement diameter, while the α_o, $(\frac{\Delta\sigma_{pr}}{\Delta\sigma_p})^2$ terms in the CEB-FIP expression are not of major significance and accounted for in the $\xi = 6.51$ for the post-tensioned beams in the author's expression.

The study reported in Ref. 18 concentrating on the area of concrete in tension and the nominal strain in the concrete at the tensile face would not give a reliable prediction of the crack width. In particular it does not account for the actual stress in the steel reinforcement and depends on measurements of strain at the concrete surface which are difficult to reliably evaluate.

Cracking in Prestressed Concrete Circular Tanks

Circular prestressed tanks are cylindrical shell elements of very large diameter in relation to their height. Hence, it is possible to treat the wall with respect to flexural cracking in a manner similar to the behavior of two-way action plates. Vessay and Preston (19) modified the Nawy expressions developed for two-way action slabs and plates (7) so that the maximum crack width can be defined as

$$w_{max} = 4.1 \times 10^{-6} \varepsilon_{ct} \, E_{ps} \sqrt{G_I} \qquad (18)$$

where ε_{ct} = tensile surface strain in the concrtee = $(\lambda_t f_p)/(E_{ps})$

f_p = actual stress in the steel

f_{pi} = initial prestress before losses

$\lambda_t = f_p/f_{pi}$

G_I = grid index = $\frac{8}{w}\left(\frac{s_2 s_1 d_c}{d_{b_1}}\right)$

s_1 = spacing of reinforcement in direction "1"

d_{b_1} = diameter of steel in direction "1"

s_2 = spacing of reinforcement in direction "2"

d_c = concrete cover to center of steel, in.

Note that w_{max} = 0.004 in. (0.1 mm) should be the allowable crack width for liquid-retaining tanks.

CRACKING AND CRACK CONTROL IN TWO-WAY ACTION SLABS AND PLATES

Flexural crack control is essential in structural floors, most of which are under two-way action. Cracks at service load and overload conditions can be serious in such floors as in office buildings, schools, parking garages, industrial buildings and other floors where the design service load and overload levels exceed those in normal size apartment building panels. Such cracks can only lead to detrimental effects on the integrity of the total structure, particularly in adverse environmental conditions.

Flexural Cracking Mechanism and Fracture Hypothesis

Flexural cracking behavior in concrete structural floors under two-way action is *significantly different* from that in one-way members. Crack-control equations for beams underestimate the crack widths developed in two-way slabs and plates, and do not tell the designer how to space the reinforcement. Cracking in two-way slabs and plates is controlled primarily by the steel stress level and the spacing of the reinforcement in the two perpendicular directions. In addition, the clear concrete cover in two-way slabs and plates is nearly constant [$\frac{3}{4}$ in. (19 mm) for interior exposure], whereas it is a major variable in the crack-control equations for beams. The results from extensive tests on slabs and plates by Nawy et al, (Ref. 3, 7, 8) demonstrate this difference in behavior in a fracture hypothesis on crack development and propagation in two-way plate action. As seen in Fig. 13, stress concentration develops initially at the points of intersection of the reinforcement in the reinforcing bars and at the welded joints of the wire mesh, that is, at the grid nodal points, thereby dynamically generating fracture lines along the paths of least resistance, namely, along A_1B_1, A_1A_2, A_2B_2, and B_2B_1. The resulting fracture pattern is a total repetitive cracking grid, provided that the spacing of the nodal points A_1, B_1, A_2, and B_2 is close enough to generate this preferred initial fracture grid of orthogonal cracks narrow in width, as a preferred fracture mechanism.

If the spacing of the reinforcing grid intersections is too large, the magnitude of stress concentration and the energy absorbed per unit grid is too low to generate cracks along the reinforcing wires or bars. As a result, the principal cracks follow diagnonal yield-line cracking in the plain concrete field away from the reinforcing bars early in the loading history. These cracks are wide and few.

This hypothesis also leads to the conclusion that surface deformations of the individual reinforcing elements have little effect in arresting the generation of the cracks or controlling their type or width in a two-way-action slab or plate. In a similar manner, one may conclude that the scale effect on two-way-action cracking behavior is insignificant, since the cracking grid would be a reflection of the reinforcement grid if the preferred orthogonal narrow cracking widths develop. Therefore, to control cracking in two-way-action floors, the major parameter to be considered is the reinforcement spacing in the two perpendicular directions. Concrete cover has only a minor effect, since it is usually small, constant in value of 0.75 in. (20 mm).

For a constant area of reinforcement determined for bending in one direction, that is, for energy absorption per unit slab area, the smaller the spacing of the transverse bars or wires, the smaller should be the diameter of the longitudinal bars. The reason is that less energy has to be absorbed by the individual longitudinal bars. If one considers that the magnitude of fracture is determined by the energy imposed per specific volume of reinforcement acting on a finite element of the slab, a proper choice of the reinforcement grid size and bar size can control cracking into preferred orthogonal grids.

It must be emphasized that this hypothesis is important for serviceability and reasonable overload conditions. In relating orthogonal cracks to yield-line cracks, the failure of a slab ultimately follows the generally accepted rigid-plastic yield-line criteria.

Crack Control Equation

The basic Eq. 1 for relating crack width to strain in the reinforcement is

$$w = \alpha \, a_c^{\beta'} \epsilon_s^{\gamma} \qquad (19)$$

The effect of the tensile strain in the concrete between the cracks is neglected as insignificant, a_c is the crack spacing, ϵ_s the unit strain in the reinforcement, and α, β', and γ are constants to be evaluated by tests. As a result of the fracture hypothesis presented, the mathematical model in Eq. 19 and the statistical analysis of the data of 90 slabs tested to failure, the following crack-control equation emerged (Ref. 3, 7):

$$w = K\beta f_s \sqrt{G_I} \qquad (20)$$

where $G_I = d_{b1} s_2 / \rho_{t1}$, is termed the grid index defining the reinforcement distribution in two-way action slabs and plates. It can be transformed in Eq. 20 to

$$G_I = \frac{s_1 s_2 d_c}{d_{b1}} \frac{8}{\pi} \qquad (21)$$

where K = fracture coefficient, having a value of $K = 2.8 \times 10^{-5}$ for uniformly loaded restrained two-way action square slabs and plates. For concentrated loads or reactions, or when the ratio of short to long span is less than 0.75 but larger than 0.5, a value of $K = 2.1 \times 10^{-5}$ is applicable. For a span aspect ratio of 0.5, $K = 1.6 \times 10^{-5}$. Units of coefficient K are in square inch per lb.

β = ratio of the distance from the neutral axis to the tensile face of the slab to the distance from the neutral axis to the centroid of the reinforcement grid (to simplify the calculations use $\beta = 1.25$, although it varies between 1.20 and 1.35)

f_s = actual average service load reinforcement stress level, or 40% of the design yield strength, f_s, ksi

d_{b1} = diameter of the reinforcement in direction 1 closest to the concrete outer fibers, in.

s_1 = spacing of the reinforcement in direction 1, in.

s_2 = spacing of the reinforcement in perpendicular direction 2, in.

1 = direction of the reinforcement closest to the outer concrete fibers; this is the direction for which crack control check is to be made

ρ_{t1} = active steel ratio in direction 1

$$= \frac{\text{area of steel } A_s \text{ per foot width}}{12(d_{b1} + 2c_1)}$$

where c_1 is clear concrete cover measured from the tensile face of the concrete to the nearest edge of the reinforcing bar in direction 1

w = crack width at face of concrete caused by flexural load, in.

Subscripts 1 and 2 pertain to the directions of reinforcement. Detailed values of the fracture coefficients for various boundary conditions are given in Table 1.

A graphical solution of Eq. 20 is given in Fig. 14 for

f_y = 60 ksi (414 MPa) and

f_s = 40% f_y = 40 ksi (165.5 MPa)

for rapid determination of the reinforcement size and spacing needed for crack control.

Since cracking in two-way slabs and plates is primarily controlled by the grid intersections of the reinforcement, concrete strength would not be of major consequence. Hence, the value of crack width in two-way action predicted by Eq. 20 should not be significantly affected if higher strength concretes are used in excess of 6,000 psi (41.4 MPa). It has to be pointed out that in two-way normal slab floors, the use of much higher strengths is not justified in economical terms.

Tolerable Crack Widths in Concrete Structures

The maximum crack width that a structural element should be permitted to develop depends on the particular function of the element and the environmental conditions to which the structure is liable to be subjected. Table 2 from the ACI Committee 224 report on cracking serves as a reasonable guide on the tolerable crack widths in concrete structures under the various environmental conditions that are normally encountered. Its values are in close agreement with the C.E.B. recommendations for most conditions of exposure.

The crack control equation and guidelines presented are important not only for the control of corrosion in the reinforcement but also for deflection control. The reduction of the stiffness EI of the two-way slab or plate due to orthogonal cracking when the limits of permissible crack widths in Table 2 are exceeded, can lead to excessive deflection both short-term and long-term. Deflection values several times those anticipated in the design, including deflection due to construction loading, can be reasonably controlled through camber and control of the flexural crack width in the slab or plate. Proper selection of the *reinforcement spacing* s_1 and s_2 in the perpendicular directions as discussed in this section, and *not exceeding 12 in. center to center*, can maintain good serviceability performance of a slab system under normal and reasonable overload conditions.

Long-Term Effects on Cracking

In most cases, the magnitude of crack widths increases in long-term exposure and long-term loading. The increase in crack width can vary considerably in cases of cyclic loading, such as in bridges but the width increases at a decreasing rate with time. In most cases, a doubling of crack width after several years under sustained loading is not unusual.

Cracking in Mass Concrete

Mass concrete denotes "concrete volumes with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of cement and attendant volume change to minimize cracking" (ACI Report 116R). Cracking in structures normally results from material and/or structural causes (3). Material cracking can be caused by shrinkage, heat of hydration, or alkali-aggregate reaction. Structural cracking is caused by external load combinations such as liquid pressure, subsidance, excessive vibrations and ambient temperatures.

In order to control such cracks, the following measures have to be taken (3):

1. Concrete with large tensile strain capacity.
2. Small daily and seasonal temperature variations.
3. Low cement content (permitted by low design stresses).
4. Cement of low heat generation.
5. Short blocks.
6. Slow rate of construction when no artificial cooling is used.
7. Low degree of restraint, as with yielding foundation, or in portions of the structure well removed from restraining foundation.
8. High yearly average temperature.
9. Absence of stress raisers, such as galleries.
10 Low placing temperature.

It is evident from this list that heat generaton during construction is one of the major factors in cracking of mass concrete systems. The most effective measures would thus entail precooling the concrete during its production and postcooling it with embedded pipe systems after it is placed.

CONCLUSIONS

The discussions presented are an overview of the rational treatment of cracking in concrete both as a material and as a structural system. Practices differ in various parts of the world due to the variability of environmental conditions as well as practices developed over the years. For cracking due to constituent material behavior, qualitative judgement procedures based on situ-experience and aggregate choice can in most cases control the crack development.

Cracking due to structural loading of concrete elements can be evaluated and predicted more concisely by computational procedures. While there is no universal agreement on the analytical expressions developed by various researchers, this paper attempts to summarize accepted approaches backed by extensive experimental research. With the aid of the expressions presented in this paper summarized in the following, the design engineer and the constructor can limit the flexural macro crack width that develops in concrete systems. By limiting the width to within the tolerable levels given in Table 2 in accordance with the prevailing environmental conditions, it would be possible to prevent or considerably minimize long-term corrosion deterioration and also maintain the aesthetic integrity of the various elements of the system.

Reinforced Concrete Beams and Thick One-Way Slabs

$$w_{max} = 0.076\ \beta\ f_s \sqrt[3]{d_c A} \times 10^{-3}$$

or

$$z = f_s \sqrt[3]{d_c A}$$

where f_s is in ksi and z not to exceed a value of 145 kip per inch for exterior exposure and 175 kip per inch for interior exposure.

Prestressed Pretensioned Beams

a) Steel reinforcement level

$$w_{max} = 5.85 \times 10^{-5} \frac{A_t}{\Sigma_o} (\Delta f_s)$$

b) Tensile face of concrete

$$w'_{max} = 5.85 \times 10^{-5} R_i \frac{A_t}{\Sigma_o} (\Delta f_s)$$

Prestressed Post-Tensioned Beams

a) Steel reinforcement level

$$w_{max} = 6.51 \times 10^{-5} \frac{A_t}{\Sigma_o} (\Delta f_s)$$

b) Tensile face of concrete

$$w_{max} = 6.51 \times 10^{-5} R_i \frac{A_t}{\Sigma_o} (\Delta f_s)$$

For nonbonded beams, the factor 6.51 becomes 6.83.

For large diameter circular prestressed concrete resevoirs $W_{max} = 4.1 \times 10^{-6}\ \epsilon_{ct}\ E_{ps} \sqrt{G_I}$

For prestressed beams with higher strength concretes, reduction multipliers λ_r and λ_o have to be applied to these expressions as follows:

$$\lambda_r = \frac{2}{(0.75+0.06\sqrt{f'_c})\ \sqrt{f'_c}} \text{ for pretensioned beams and}$$

$$\lambda_o = \frac{2}{0.75+0.06\ \sqrt{f'_c}} \text{ for post-tensioned unbonded beams.}$$

Two-way Action Structural Slabs and Plates

$$w_{max} = K\beta f_s\ \sqrt{G_1}$$

where
$$G_1 = \frac{s_1 s_2 d_c}{d_{b1}} \times \frac{8}{\pi}$$

The values of coefficient K are as follows

K	Fully restrained slabs and plates
2.8×10^{-5}	Uniformly loaded, square
2.1×10^{-5}	At concentrated loads and columns
2.1×10^{-5}	$0.5 > \ell_s/\ell_\ell < 0.75$
0.6×10^{-5}	$\ell_s/\ell_\ell < 0.5$

*For simply supported slabs multiply above values by 1.6. interpolate K values for intermediate span ratios ℓ_s/ℓ_ℓ or for partial restraints at the boundaries such as cases of end and corner panels of multipanel floor systems.
ℓ_s and ℓ_ℓ are respectively the short and long spans of the two-way slab or plate.

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METRIC (SI) UNIT EQUIVALENTS

1 inch	= 25.4 mm
1 foot	= 0.305 m
1 in.^{2}	= 645.16 mm^2
1 in.^{3}	= 16,387.06 mm^3
1 in.^{4}	= 416.231 mm^4
1 psi	= 6.895 Pa
1 ksi	= 6.895 MPa
1 lb	= 4.448 N
1 kip	= 4448 N
1 lb/ft	= 14.594 N/m
1 kip/ft	= 14.594 kN/m
1 kip-in	= 113 N-m
1 $\sqrt{f'_c}$ psi	= 0.083036 $\sqrt{f'_c}$ MPa

NOTATION

A_g = gross area of concrete, in.^{2}

A_s = area of tension reinforcement, in.^{2}

A_t = the effective concrete area in uniform tension, in.^{2} In the case of two-way slabs and plates, it is = 12 (d_{b_1} + $2c_1$) for a 12 in. strip, defining the concrete area stretched in tension, in.^{2}

a_{cs} = stabilized mean crack spacing in.

CEB = Comite Euro-International du Beton

c = clear concrete cover, in.

d_b = diameter of reinforcing bar or wire, in.

d_c = effective depth to center of the first layer of reinforcement, in.

E_c = modulus of elasticity of concrete, psi, ksi

E_s = modulus of elasticity of steel, psi, ksi

f'_c = cylinder compressive strength of concrete, psi

f_y = yield strength of the reinforcement by the 0.2% offset, ksi

f_s = service load level steel stress = 0.6 f_y in beams and 0.4 f_y in two-way action slabs and plates, ksi

f'_t = tensile splitting strength of concrete (ASTM C 496-62T), psi

F = prestressing force, lb., kip

f_d = stress in the prestressing steel corresponding to the decompression load, ksi

f_{nt} = stress in the prestressing steel at any load level beyond the decompression load, ksi

f_r = modulus of rupture of concretes, psi

G_{I1} = grid index in direction "1" (= $d_{b1}s_2/\rho_{t1}$) closest to outer

G_{I2} = grid index in direction "2" $(d_{b2}\, s_1/\rho_{t2})$ in direction perpendicular to "1", in^2

h_1 = distance from the neutral axis to the centroid of the reinforcement, in.

h_2 = distance from the neutral axis to the tension face, in.

h = total slab of plate thickness, in.

K = fracture coefficient dependent on load or reaction type and on the support condition

k = kip, 1000 lb.

ksi = kips per sq. inch

Δf_s = net stress in the prestressing steel or the magnitude of tensile stress in the mild steel after decompression, ksi

λ_r, λ_o = modifying reduction multipliers for crack width in higher strength prestressed beams

ρ = steel percentage A_s/bd in beams

ρ_{t1} = active steel ratio

$$= \frac{\text{area of steel } A_s \text{ per foot width}}{12(d_{b1} + 2c_1)}$$

R_1, β = Distance ratio h_2/h_1 = ratio of distance from neutral axis to tensile face of concrete to distance from neutral axis to the center of gravity of reinforcement

s = spacing of reinforcement, in.

w_{max} = maximum crack width in inches at concrete face, in.

"1" = direction of reinforcing elements closest to outer concrete tensile fiber, for which crack control check is to be made

α, β', τ = coefficients of regression function in slabs

Σo = sum of perimeters of all effective bars and tendons crossing the section on the tension side, in.

T = total tensile force transferred by bond from steel, lb., kip

μ = bond strength in concrete, psi

w = maximum crack width at steel level, in.

y_t = distance from the centroidal axis of gross section to the extreme fiber in tension, in.

Z = quantity limiting distribution of flexural reinforcement in beams and thick one-way slabs = $f_s\sqrt[3]{d_c A}$, kip/inch

TABLE 1 FRACTURE COEFFICIENTS FOR SLABS AND PLATES

Loading type[a]	Slab shape	Boundary condition[b]	Span ratio,[c] S/L	Fracture coefficient 10^{-5} K
A	Square	4 edges r	1.0	2.1
A	Square	4 edges s	1.0	2.1
B	Rectangular	4 edges r	0.5	1.6
B	Rectangular	4 edges r	0.7	2.2
B	Rectangular	3 edges r, 1 edge h	0.7	2.3
B	Rectangular	2 edges r, 2 edges r	0.7	2.7
B	Square	4 edges r	1.0	2.8
B	Square	3 edges r, 1 edge h	1.0	2.9
B	Square	2 edges r, 2 edges h	1.0	4.2

[a] Loading type: A, concentrated; B, uniformly distributed.

[b] Boundary condition: r, restrained; s, simply supported; h, hinged.

[c] Span ratio S/L: S, clear short span; L, clear long span.

TABLE 2. MAXIMUM TOLERABLE FLEXURAL CRACK WIDTHS

Exposure condition	Crack width in.	Crack width mm.
Dry air or protective membrane	0.016	0.40
Humidity, moist air, soil	0.012	0.30
De-icing chemicals	0.007	0.18
Seawater and seawater spray; wetting and drying	0.006	0.15
Water-retaining structures (excluding nonpressure pipes)	0.004	0.10

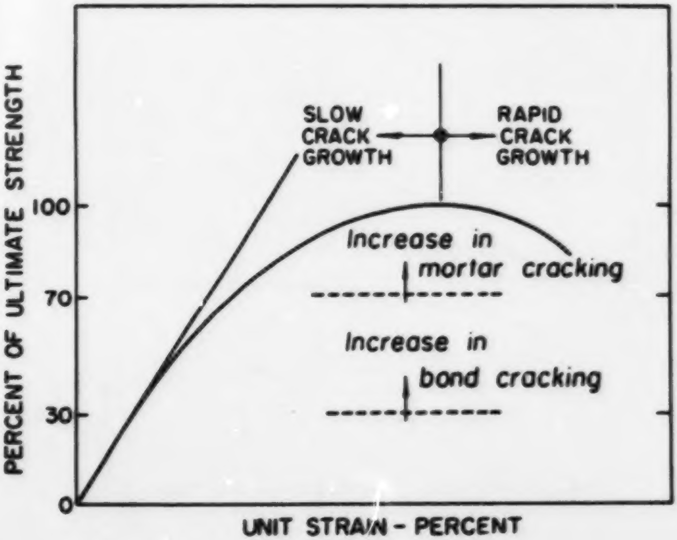


Fig. 1. Schematic Stress Strain Diagram of Concrete in Microcracking

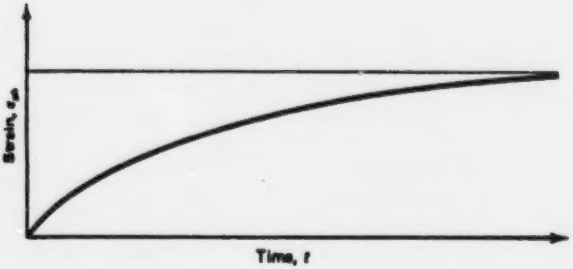


Fig. 2. Shrinkage strain vs. Time

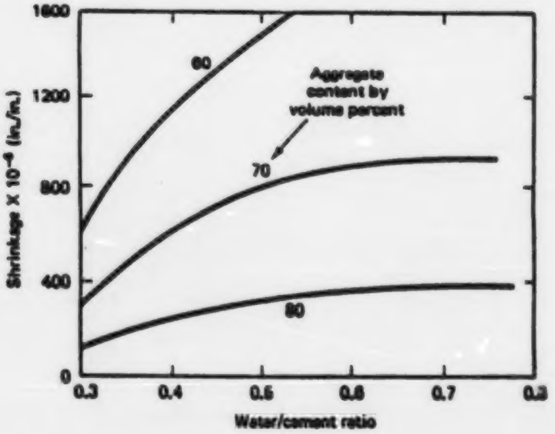


Fig. 3. Shrinkage Coefficient vs. W/C Ratio

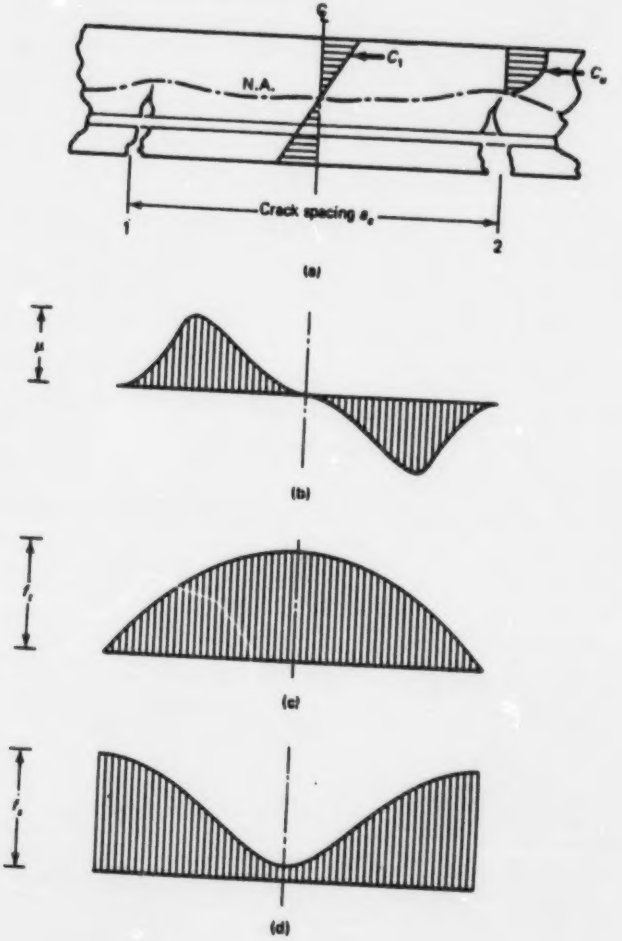

℄
C_1
C_s
N.A.
Crack spacing a_c
1
2
(a)
μ
(b)
f_c
(c)
f_s
(d)

Fig. 4. Schematic Stress Distribution Between Two Flexural Cracks

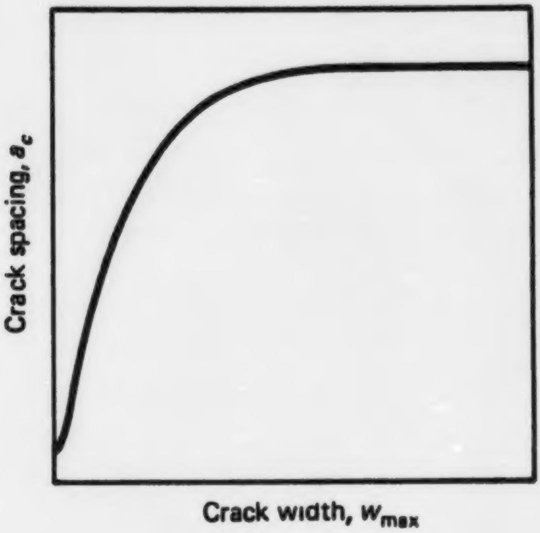


Fig. 5. Schematic Variation of Crack Width with Crack Spacing

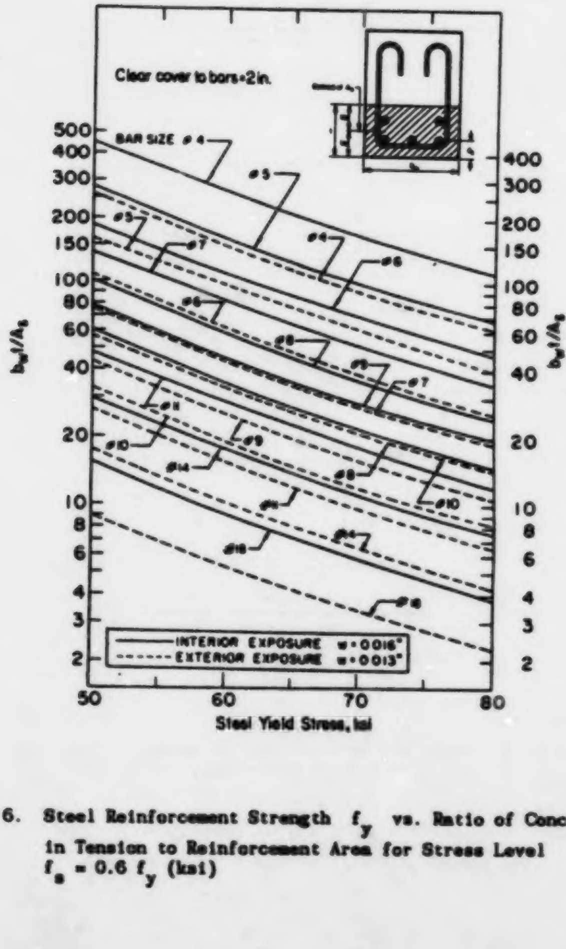


Fig. 6. Steel Reinforcement Strength f_y vs. Ratio of Concrete Area in Tension to Reinforcement Area for Stress Level $f_s = 0.6 f_y$ (ksi)

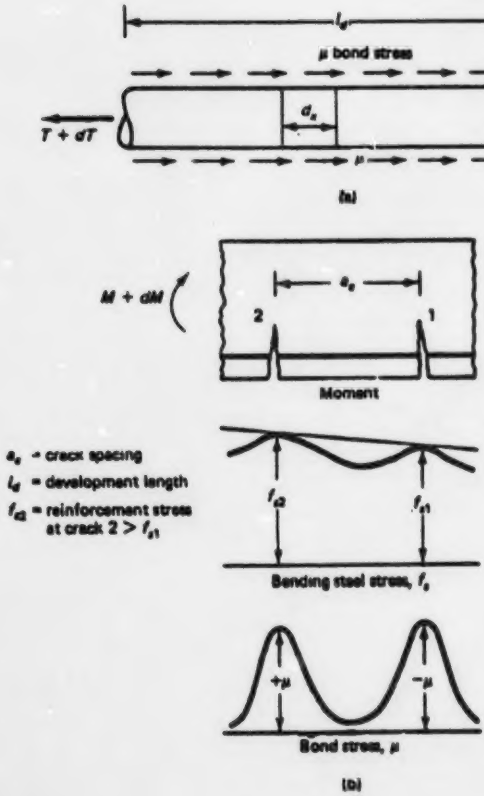
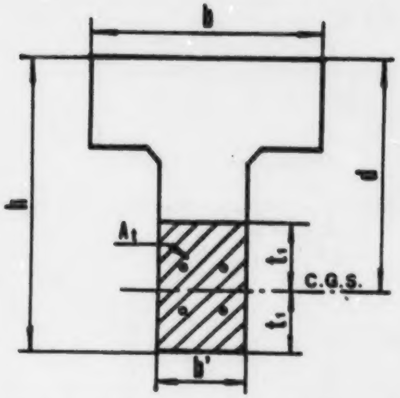
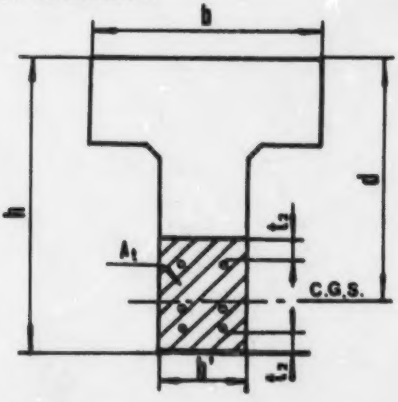

l_d
μ bond stress
d_x
T + dT
T
μ
(a)
M + dM
M
a_c
2
1
Moment
a_c = crack spacing
l_d = development length
f_{s2} = reinforcement stress at crack 2 > f_{s1}
f_{s2}
f_{s1}
Bending steel stress, f_s
+μ
−μ
Bond stress, μ
(b)

Fig. 7. Force and Stress Distribution in a Stabilized Crack in a Prestressed Beam


b
h
d
A_t
t_1
C.G.S.
t_1
b'

(a) FOR EVEN DISTRIBUTION OF REINFORCEMENT IN CONCRETE


b
h
d
t_2
A_t
C.G.S.
b'
t_2

(b) FOR NON-EVEN DISTRIBUTION OF REINFORCEMENT IN CONCRETE

Fig. 8. Effective Concrete Area in Tension. (a) Even Reinforcement Distribution (b) Noneven Reinforcement Distribution

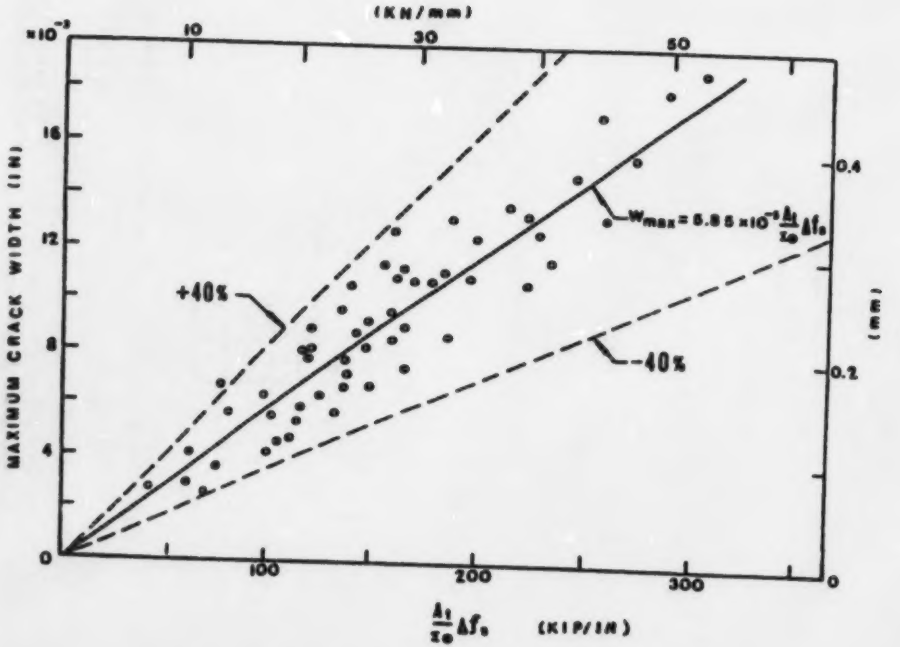


Fig. 9. Linearized Maximum Crack Width vs. $(\frac{A_t}{\Sigma_o}) \Delta f_s$

(Pretensioned Beams)

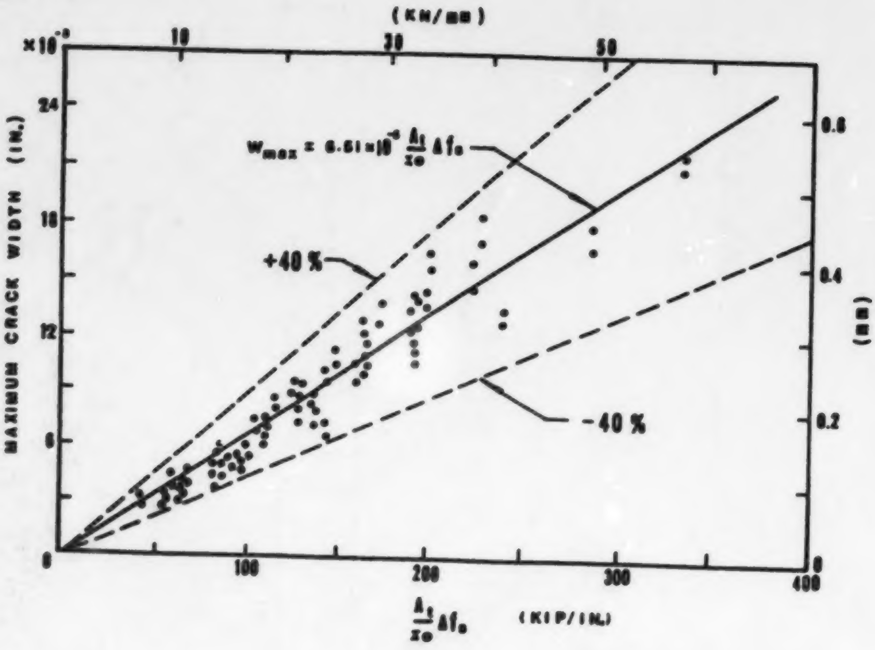


Fig. 10. Linearized Maximum Crack Width vs. $(\frac{A_t}{\Sigma o})\ \Delta f_s$
(Post-Tensioned Beams)

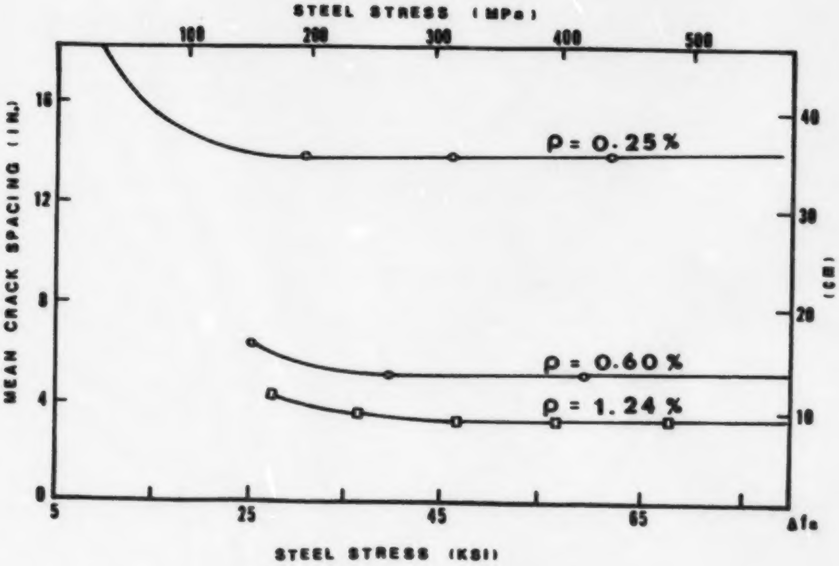


Fig. 11. Effect of Steel Percentage on Mean Crack Spacing in Prestressed Beams

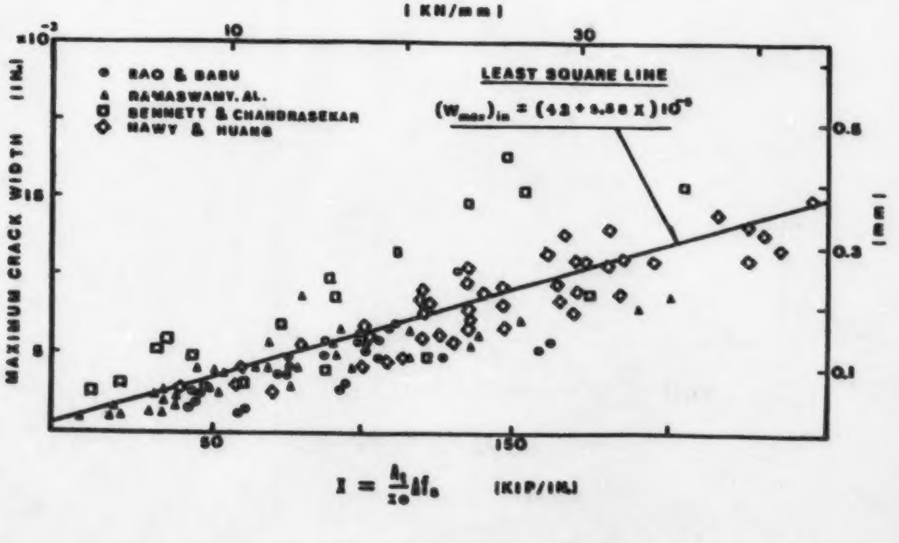


Fig. 12. Reinforcement Stress vs. Crack Width (Best Fit Data of Several Investigators)

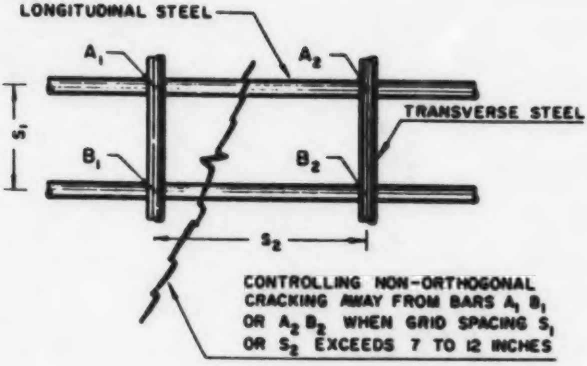
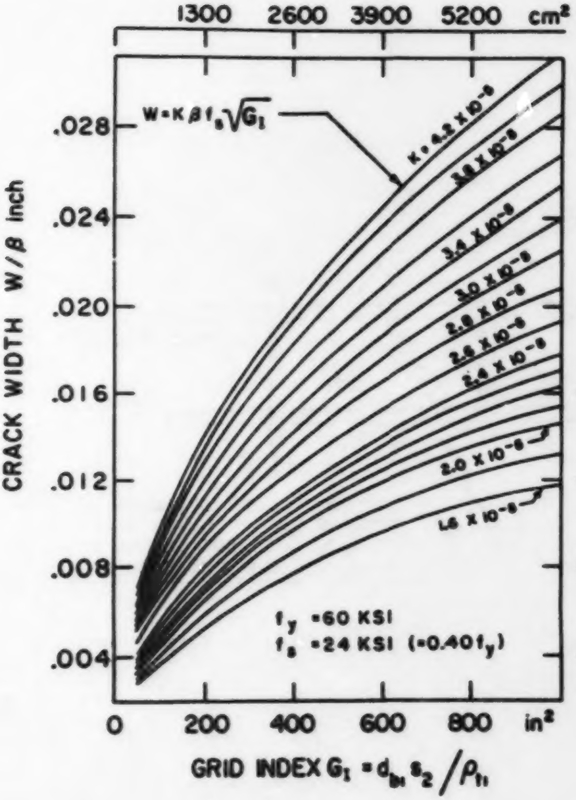

LONGITUDINAL STEEL
A_1
A_2
TRANSVERSE STEEL
S_1
B_1
B_2
S_2
CONTROLLING NON-ORTHOGONAL CRACKING AWAY FROM BARS A_1 B_1 OR A_2 B_2 WHEN GRID SPACING S_1 OR S_2 EXCEEDS 7 TO 12 INCHES

Fig. 13. Grid Unit in Two-Way-Action Reinforcement



$$= \frac{s_1 s_2 d_c}{d_{b1}} \cdot \frac{8}{\pi}$$

Fig. 14. Crack Control Reinforcement Distribution in Two-Way-Action slabs and plates for all exposure conditions: $f_y = 60$ ksi. $f_s = 24$ ksi ($= 0.40 f_y$)

INNOVATIONS IN THE TESTING OF FRESH AND HARDENED CONCRETE

by

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Abstract

Three new devices for testing fresh concrete are described briefly in this paper. The three innovations are the K-Slump tester, the K-Mini Air meter, and the K-5 Accelerated Strength tester. Several experiments were carried out in the laboratory and in the field to correlate the performance of these devices with other ASTM accepted testers. The results indicate that the new devices correlated very well with the slump cone, the volumetric and the pressure airmeters, and the accelerated strength boiling water method. However, the new devices appear to be simpler, easier, superior, and more efficient and economical to use than the other standard test methods.

Keywords: Compressive strength; concrete; K-5 accelerated strength tester; K-Mini Air Meter; K-slump tester; pin penetration test.

INTRODUCTION

Concrete is one of the most economical engineering materials which is used in all types of construction and repair works. It is also a very friendly material and many people are prepared to use it even though they know very little about its properties except that it can be mixed and placed easily to fill a specific form and will harden shortly thereafter. It is imperative that the friendly and economic characteristics of concrete should be matched by simple, easy, and accurate testing procedures that can be used to monitor the quality of concrete production. Hence, this paper gives a brief description of four new, efficient, and easy innovations in testing of fresh and hardened concrete.

SLUMP AND WORKABILITY TESTERS

Many of the properties of hardened concrete, such as strength, durability, and deformation are affected by its workability and slump. Although there is general agreement about the importance of workability, yet there is no unique and universal test which is accepted as a true measure of workability. However, the slump test, which was introduced in the early part of the twentieth century, is widely used as an indicator of workability. The main advantages of the slump cone are its simplicity and its sensitivity to the water content and the consistency of a specific mixture. Some of its disadvantages are that it is cumbersome to carry around and it is awkward to use in performing the test. Hence, several devices were developed to measure workability and they are described in detail by others (1,2,3,4,5). In this section, the K-slump apparatus will be highlighted and the results of recent tests will be described.

K-Slump Tester

The new K-Slump (4) tester is presented in Fig. 1. This tester can be used to measure the slump and the workability of fresh concrete in 1 minute. The main components and dimensions of the tester are outlined below:

1. The main component is a chrome plated steel tube with external and internal diameters of 19 and 16 mm (0.75 and 0.63 in.) and a total length of 250 mm (9.84 in.). The lower part of the tube is 155 mm (6.10 in.) long and contains two types of openings and a solid cone at its end to facilitate its insertion in the concrete; four openings are rectangular slots 51 mm (2.01 in.) long and 8 mm (0.31 in.) wide while the rest are 22 round openings, with 6.4 mm (0.25 in.) in diameter. The openings are distributed uniformly in the lower part of the tube so that they will let the mortar part of the mix flow into the tube.

2. The second component is a disc floater 60 mm (2.36 in.) in diameter and 2.4 mm (0.09 in.) thick and it divides the tube into two parts: the upper part serves as a handle to insert the tester in the concrete while the lower part serves as the testing component. The disc floater is a guide to indicate the level to which the tester should be inserted into the concrete; it also serves the purpose of preventing the tester from sinking into the concrete.

3. The third component is a hollow plastic rod 13 mm (0.51 in.) in diameter and 250 mm (9.84 in.) long which contains a graduated scale marked in cm from 0 to 11. The rod can move freely inside component one and can be used to measure the height of the mortar inside the tube. Both ends of the rod are plugged with plastic caps to prevent any material from damaging the scale.

4. The fourth component is an aluminum cap 30 mm (1.18 in.) in diameter and 2.25 mm (0.89 in.) long, which is fitted with a set screw that can be used to line up the zero reference of the scale.

   In the upper part of component one, there is a small pin that projects slightly inside the tube so as to support the measuring component when it is not in use.

   The total weight of the apparatus is 226 g (0.50 lb.).

The procedure to measure the slump and workability of the concrete with the new apparatus is to partly insert the tester into a sample of freshly-mixed concrete to the preset level of the disc floater as shown in Fig. 2. Next, allow the mortar portion of the concrete to penetrate through the openings of component one for a period of 60 seconds and measure the height of the mortar penetration into the tube by means of the floating rod protruding above the top of the hollow tube. The larger the slump of the concrete, the higher the reading on the scale of the measuring rod. This first reading will be designated as the K-slump of the concrete. If desired, a second reading will be taken by raising the measuring rod, and then removing the tester from the concrete and allowing the mortar in the tube to seek its new level as shown in Fig. 3. This second measure will be referred to as the workability index. The first reading or the K-slump is a function of the flowability and consistency of the mixture while the second reading is a reflection of the cohesiveness and segregation proneness of the same mixture.

Experimental Investigations

Two sets of tests were performed with the new apparatus. The first one was a round robin in five private and government laboratories in the U.S.A. The second set of tests was performed at the University of Saskatchewan. Details and results of the tests will be briefly outlined, and discussed separately.

Tests in the United States of America

Five laboratories performed the tests and they will be designated A, B, C, D, and E; in total they performed 420 tests. The results from Laboratory B will be discussed first as they were made on the same concrete mixture. Statistical analysis of the results is presented in Table 1.

The analysis in the table indicates that the difference between the means of the slump (6) and the K-slump is about 1/4 while the difference between their standard deviation is about 1/2. The t-test at the 95% level of confidence indicated that the two testers give basically the same result: t(calc.) = 1.684; t(tab) = 1.998

The mean of the workability index W is 3.06 and its standard deviation is 0.67. This indicates that the workability of the tested mix was good since both its K-slump and workability index had close values.

The mixes that were tested by Laboratories A, C, D, and E covered the following wide range of variables:

Cement Content: 190 to 469 kg/m^3 (320 to 790 lb/yd^3)

Coarse Aggregate: gravel, crushed rock, lightweight aggregates.
Maximum size 10 to 38 mm (3/8 to 1 1/2 in.)

Admixtures: many types

Air content: 1 to 8.5 percent

Slump: 10 to 200 mm (1/2 to 8 in.)

The four laboratories performed 384 tests and their results for slump and K-slump are plotted in Fig. 4. Analysis of the data gives the following linear equation.

$$S = 1.08 + 0.69K \qquad (1)$$

Where S = Slump in., measured by the standard cone
K = K-Slump cm, measured by the new tester

The coefficient of correlation is 0.85. Non-linear equations could have given better correlation, but the simplicity of the linear equation outweighs the advantages of the other equations. The standard error of the linear equation is about 3/4 units. As may be seen from Fig. 4 and equation (1) the new tester is not sensitive enough for low slump concretes and hence, it should be used with concretes having a slump of over 25 mm (1 in.). However, it is of interest to note that when the slump is in the range of 3 to 4 in. (76 to 102 mm) the K-slump will range between 3 1/4 and 3 3/4 units. The same observation was made by Bickley*, who used the K-slump tester to control the quality of concrete for the CN Tower in Toronto, Canada, the highest free standing structure in the world (1905 ft., 550 m).

The relationship of workability index and slump for the data from laboratories A, B, C, and E are given in Fig. 5 and equation (2).

$$W = 0.73 + 1.47S - 0.12S^2 \qquad (2)$$

Where W = Workability index in cm, measured by the new tester

S = Slump in inches, measured by the standard cone

The standard error of this relation is also about 3/4 units. Since the curve is parabolic, an optimum value for workability index with least internal friction and segregation is calculated to be 3.87 which corresponds to a slump of 6.25 in. (160 mm).

Tests at the University of Saskatchewan

These tests were carried out to determine the reliability of the K-Slump tester using methods similar to the ones described in the ASTM C 802 and C 670.
The tests were performed by four independent operators, thus simulating an interoperator study. Four concrete mixtures were prepared using portland cement Type I and normal weight aggregates. Each mixtures contained the same amount of materials except for a different water content as outlined in Table 2.

Local crushed aggregate of 1 in. (25 mm) maximum size were used in all mixtures. An air-entraining agent of vinsol resin conforming to ASTM C 260 was added to the mix. The aggregate-cement, and the coarse-fine aggregate ratios were kept constant at 7.0 and 1.19, respectively. The water content was increased to obtain different consistencies. The resultant water-cement ratios were 0.50, 0.56, 0.62, and 0.67.

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* Private communication from John A. Bickley, Consulting Engineer, Ontario, Canada.

A pan mixer was used to mix the concrete before it was placed in separate plastic pails by each operator. The pails were rocked lightly to level the surface of the concrete before inserting the K-slump tester therein. The results of the tests are given in Table 3.

The data under headings a, b, and c are the replicate measurements for each operator's given material: A, B, C, and D. All measurements of the K-slump are in mm (10 mm = 0.39 in.) and correspond approximately to the slump measured by the cone in inches. In Fig. 6, the consistency of the materials arranged in ascending order was plotted for each of the operators, and this shows a similar pattern was obtained by all operators. Although this research is not an interlaboratory one, an indication of the indexes of precision can be calculated as described in ASTM C 670. The results of the calculations are given in Tables 4 and 5. Note that the average maximum range of three measurements by a single operator is 16 mm, while that for multioperators is 18 mm. Similarly, the average maximum difference between two averages of three measurements by a single operator is 8 mm, while that for multioperators is 9 mm. Of course a slightly larger value would be expected with a fresh concrete that has small consistencies, and vice versa.

Conclusions

The following conclusions can be made from the tests:

1. There is good correlation between the slump measured by the standard cone and the K-Slump.
2. A consistent response was obtained testing the different materials by single or multioperators.
3. The standard deviation remained essentially constant with an increase in the consistency of the materials.
4. The coefficient of variation decreased as the consistency of the material increased.

5. The maximum difference between two averages of three measurements by a single operator was 8 mm, while that for multioperators was 9 mm.
6. The maximum range of three measurements by a single operator was 16 mm, while that for multioperators was 18mm.
7. The K-Slump tester is a new efficient and simple device that can measure the slump and workability of freshly mixed concrete in 60 seconds.

AIR METERS

Air-entraining admixtures can be incorporated into concrete to improve the consistency and cohesiveness of the fresh mixture, as well as the durability of the hardened material. A good air-void system in hardened concrete is of paramount importance in resisting seawater, de-icing salts, and in particular freezing and thawing conditions. In order to insure good durability, it is important to provide fine and well-distributed air voids in hardened concrete. The air-void system can be determined by microscopic analysis on hardened concrete where the total air content and specific surface can be measured and the spacing factor can be calculated.

At this point in time, there is no standard test that can permit the examination of the air-void size and distribution in fresh concrete. However, the engineering profession has so far accepted a de facto proposition that the void sizes are small in air-entrained concrete and that the durability of the hardened concrete will be satisfactory provided the fresh concrete contains the prescribed amount of air.

The air volume in fresh concrete can be measured at the job site using either the volumetric method [ASTM C 173] or the pressure method [ASTM C 231] (6). If the air content meets the design specification, the concrete is accepted; however, if it does not, theoretically, the concrete is rejected. Therefore, it is important to develop some level of confidence as to how air volume measurements in fresh concrete relate to those observed in hardened concrete.

Air Content Measurements

The testing program consisted of developing several concrete mixtures of various materials and mix proportions. The air content of each fresh concrete was measured using two volumetric airmeters and one pressure airmeter (Fig. 7). For the hardened concrete, a linear traverse procedure was chosen. A brief outline of these test devices is given below.

Pressure Airmeter

The total air content was measured by a pressure airmeter (ASTM C 231, Type A device). The test consists of applying a predetermined air pressure over a fresh concrete sample located inside a sealed container with water on top of it. The pressure compresses the air bubbles in the concrete sample and causes a drop in a pressure gage, which is calibrated to read the total air volume contained in the concrete sample. Since the applied pressure can force water and air into aggregate voids, this method is limited to concrete containing dense aggregate. The results obtained with this method are sensitive to the degree of aggregate saturation, the magnitude of the applied pressure, and the changes in barometric pressure. The pressure airmeter employed in this research was calibrated to measure air volumes up to 7.5% by applying 0.103 MPa (15 psi) air pressure. Whenever the air volumes exceeded 7.5%, lower pressure values of 0.034 or 0.069 MPa (5 or 10 psi) were exerted, and correction factors were used to obtain the air content.

Roller Airmeter

The total air content was also measured by a roller airmeter (ASTM C 173). The test consists of pouring water over a concrete sample, then sealing the chamber and agitating the water to free the air from the fluid concrete. The rising air causes a drop in the water level at the tapered neck of the apparatus, which is calibrated to reflect the percentage of air content. This test does not result in water migration into aggregate voids, and it is therefore suitable for concrete made with either dense or porous aggregate.

Mini Airmeter

This is a new volumetric apparatus (7). The device is portable and easy to use. Research indicates that this device is capable of accurately measuring the air content of normal-weight and lightweight aggregate concretes. The mini airmeter consists of two basic components: (1) a small container measuring 76 mm (3 in.) in diameter and 51 mm (2 in.) in length which holds a sample of fresh concrete: (2) a large cylindrical container with a conical cover and an extension pipe at its apex. The small container holding the concrete sample is placed inside the bottom portion of the larger container, which is then filled with water. The assembly, which is about one tenth the volume of the roller meter, is sealed and shaken in order to release the air from the fresh concrete. Isopropyl alcohol is then added to fill the chamber and dispel any foamy material from the water surface. The volume of added alcohol is used to calculate the air content in the concrete.

Linear Traverse Apparatus

The air content in hardened concrete was determined microscopically using a linear traverse apparatus [ASTM C 457]. This apparatus consists of a stereoscopic microscope and a hand-driven linear traverse platform with adequate lighting. The polished hardened concrete specimen is placed on the platform and moved smoothly and uniformly while observing its surface with the microscope. The light beam is adjusted on the specimen surface at such an angle that the air voids are clearly demarked by a shadow. The platform is then moved by means of calibrated screw, and measurements of the air voids are taken by means of special counters

Mixture Characteristics

A wide range of materials and mix proportions were adopted for developing 21 concrete mixtures. ASTM Type

I/II portland cement was used in all mixtures. The cement content ranged between 267 to 540 kg/m^3 (450 to 910 lb/yd^3). A powder silica fume was added to each of the high-strength concretes. Normal-weight rounded aggregates from gravel deposits were used in 17 mixtures. The nominal aggregate size was either 10 mm (3/8 in.) or 25 mm (1 in.). In four of the concretes, an expanded-shale aggregate with a nominal size of 10 mm (3/8 in.) was used. A natural river sand with a fineness modulus of 3.1 was used for the fine aggregate.

For normal-weight concretes, the approximate water-cementitious material ratio (W/CM) was 0.23, 0.34, or 0.50. These ratios were chosen so that the experimental program would cover concretes with high, medium, and low-strength levels, respectively. All lightweight concretes had a fixed W/CM of 0.30.

A melamine-based superplasticizer was incorporated into each of the concrete mixtures. A vinsol resin air-entraining admixture was used to obtain air contents that varied between 1.5 to 12.5%. In two concretes, an antiwashout admixture and an air-detraining additive were used to yield high-viscosity fluid mixtures typical of some underwater concretes. The mixture proportions of the 21 concretes are listed in Table 6.

Tests Performed at the University of California, Berkley, California (8)

All concretes were mixed and tested for fresh air content at the laboratory in a controlled environment where the temperature was maintained at 23 ± 2° C (73 ± 3° F) and the relative humidity was 97 ± 3%. The concrete was mixed in a 0.028-m^3 (1-ft^3) open-pan mixer. The total volume of the ingredients per mix was 0.014 m^3 (0.50 ft^3). The charging sequence of concrete materials consisted of adding sand and gravel to the mixer along with one third of the mixing water and allowing them to mix for 1 min. before adding the cementitious materials and another third of the mixing water. Powder antiwashout and air-detraining admixtures were preblended with the cementitious materials. Other liquid additives were diluted with the remaining water and added last. The concrete was mixed for 3 min; then after 3 min of resting, it was remixed for an additional 2 min.

At the completion of mixing, the consistency of the concrete was evaluated using the K-slump tester. The unit weight and air content of the fresh concrete were then measured within 15 min from the completion of mixing. Each test was conducted by the same individual in order to minimize operator-related errors. The air contents of normal-weight concretes were measured with the pressure, roller, and mini airmeters; however, only the roller and mini airmeters were employed to measure the air contents of lightweight concretes.

From each concrete, three cylinders measuring 76 mm (3 in.) in diameter and 152 mm (6 in.) in length were cast 15 to 20 min following the completion of mixing. The specimens were stored in a fog room (97 + 3% relative humidity) that was maintained at 23 ± 2°C (73 ±3°F). After two days of curing, the specimens were stripped and returned to the fog room.

Two cylinders from each concrete were tested in compression after 28 days of curing. The remaining cylinder was used for air void determination, which was carried out after approximately 56 days of moist curing. Each specimen was sliced longitudinally in the middle perpendicular to the top and bottom surfaces so as to provide a plate measuring 76 mm (3 in.) in width, 152 mm (6 in.) in length, and 25 mm (1 in.) in thickness. Each plate was ground on both sides with successively finer abrasives. Both sides of the polished plate were examined microscopically. The length of the traverse was either 1905 mm (75 in.) or 2413 mm (95 in.) depending upon the nominal aggregate size in the concrete, in accordance with ASTM C 457.

Test Results and Discussion

The measured K-slump and unit weight values of fresh concrete are listed in Table 7. The K-slump values ranged between 1.25 and 11. The fresh unit weight values of the 17 normal-weight concretes varied between 2,210 and 2,488 kg/m^3 (138.1 and 155.5 lb/ft^3), while those of lightweight concretes were 1,920 to 1,139 kg/m^3 (120.0 to 123.9 lb/ft^3). The calculated gravimetric air contents [ASTM C 138], and the measured air contents of fresh and hardened concretes are listed in Table 7. Similarly, the 28-day compressive strength values of all tested samples are listed in Table 7.

Figure 8 compares the air contents of fresh concretes measured using the pressure, roller, and mini airmeters to those determined by the linear traverse method on hardened concrete. Linear regression analysis was used to correlate the air contents of fresh concrete measured with the three different airmeters to those determined microscopically. The analysis indicates that linear relationships exist between the air content measurements made with the pressure, roller, and mini airmeters and those determined using the linear traverse method, as shown in Fig. 8. These linear expressions can be written as follows:

$$H = 1.10 + 0.58\ P \qquad (1)$$
$$H = 0.42 + 0.78\ R \qquad (2)$$
$$H = 0.43 + 0.74\ M \qquad (3)$$

where

H = the mean percent air content determined using the linear traverse method, and

P, R, and M = the air contents of fresh concrete measured using the pressure, roller, and mini airmeters, respectively.

Equations 1, 2, and 3 had correlation coefficients of 0.958, 0.987, and 0.976, respectively.

The air volumes of fresh concrete were consistently greater than those determined for hardened concretes for air volumes exceeding 4%. The ratios of fresh to hardened concrete air contents were: 1.22, 1.13, and 1.19 for the pressure, the roller and mini airmeters respectively. Their corresponding coefficients of variation were: 21, 10, and 11%.

Could the discrepancy between air contents of fresh and hardened concrete be due to a combination of settlement and shrinkage in concrete which takes place after casting or due to different levels of consolidation between molded concrete specimens and those placed in the airmeters? A satisfactory answer for this question will have to wait for further research in this area.

Conclusions

For the investigated materials and mixture proportions, the following conclusions can be drawn:

1. The test results obtained by the new Mini Air meter correlated well with those obtained using the other meters; and the Mini Air meter can measure the air content of fresh concrete in a simple and efficient manner.

2. Test results from microscopic analysis on hardened concrete using the linear traverse method do not coincide with those measured using the pressure, roller, or mini airmeters on fresh concrete. The air content in fresh concrete is slightly igher than that observed in hardened concrete, especially when the air content exceeds 4%.

3. On the average, the estimated air volume of fresh concrete is 10 to 20% greater than that determined microscopically on the hardened concrete. This corresponds to approximately 0.5 to 1.5% additional air for the range of air volumes typically specified in construction.

4. The discrepancy between the air measurements of fresh and hardened concretes is lower when the air content is monitored using the roller or mini airmeter instead of the pressure airmeter. The greater deviation associated with the latter

apparatus may decrease if the correction factors used to estimate high air volumes are frequently verified.

5. The air content of fresh concrete deviates more from that of hardened concrete as the compressive strength of the concrete decreases and the air content increases.

6. A close correlation can be derived between air contents obtained with a pressure airmeter on fresh concrete (P) and those determined by the linear traverse method on hardened concrete (H). The relationship relating the two values can be expressed as

$$H = 1.10 + 0.58\ P$$

7. Similar relationships exist between air determinations made using either the roller airmeter (R) and the mini airmeter (M) and air volumes of hardened concrete. These relationships can be written as

$$H = 0.42 + 0.78\ R$$
$$H = 0.43 + 0.74\ M$$

8. The new mini airmeter is accurate, simple to operate, and practical for use in the field and the laboratory.

K-5 ACCELERATED STRENGTH TESTER

CANMET has been one of the pioneers as well as one of the proponents of accelerated testing of concrete (9). Following in that tradition, they have used the K-5 accelerated-strength tester to investigate its performance under two conditions: testing air and non-air-entrained concrete; and relating its performance to Method B of ASTM C 684, using concrete that contained entrained air and 25 percent fly ash by weight of cement. The fly ash was obtained from different sources and did not have identical physical and chemical properties.

The mixture for the first set of tests contained Type I cement and the mix proportions and the properties of the fresh concrete are given in Table 8. The concrete was cured and tested in accordance with standard ASTM Methods C 39 and C 192 at the age of 28 days; companion specimens were also cured and tested with the accelerated K-5 strength tester.

Description of the K-5 tester was given in previous papers (10), but for the sake of clarity the following summary of the method is presented here: the concrete in this method is placed in 3 x 6-in. (75 150-mm) cylinders, as shown in Fig. 9, and exposed for 3 hr to a pressure of 1500 ± 15 psi (10.3 ± 0.1 MPa) and an elevated temperature of 300 ± 3 F (149 ± 1.5 C). Then it is cooled for 2 hr before it is tested. The total curing period for the concrete is 5 hr after making the cylinders. The results of the CANMET testing program are given in Fig. 10 and 11 and they relate the standard strength at the age of 7 and 28 days to the 5-hr accelerated strength. Best-fit linear lines were calculated and superimposed over the data in the same figures. A summary of the equations and their coefficients of variation are given in Table 9. As can be seen from the table and the graphs, the results are compatible, and the relationship between the two procedures is satisfactory.

The concrete for the second set of tests contained fly ash from different sources, and 25 percent of the cement was replaced by the fly ash (weight basis). The mix proportions and the properties of the fresh concrete are given in Table 10. Accelerated curing of the concrete was carried out either by the boiling method B or by the K-5 procedure of ASTM C 684. Method B consists of curing the concrete in boiling water for 3 1/2 hr after curing it under standard conditions for 23 hr. A summary of the test results is given in Table 11 and Fig. 12 and 13. Linear best-fit lines were calculated and superimposed over test data in the figures; the equations and their coefficients of variation are given in Table 12. The results indicate very good correlation between both the B and K-5 methods and the standard strength tests at the ages of 7 and 28 days. Fig. 14 is further proof of the good correlation between both the B and K-5 methods of ASTM C 684.

Other Data

Philleo* (11) provided the author with data that were obtained during an interlaboratory round robin for accelerated strength testing for ASTM C 684. The variance and the coefficient of variation for that data were calculated to correlate these with corresponding values determined from data that were obtained with the K-5 method. It should be noted that the interlaboratory data were obtained from two types of concrete that contained either Type I or Type III cements, while the data from the K-5 method were obtained for concrete with many more variables. These variables included Type I, III, and V cements, light and ordinary weight aggregates, fly ash, air-entraining, calcium chloride, and water-reducing admixtures. Also, the size of the cylinders for the interlaboratory tests were 6 x 12 in. (150 x 300 mm), while those for the K-5 tests were 3 x 6-in. (75 x 150-mm) cylinders. The results of the calculations are summarized in Table 13.

As can be seen from Table 13, the coefficient of variation of the strength in Methods A, B, and C for the 6 x 12-in. (150 x 300 mm) cylinders ranged between 14 and 23 percent with a mean of 18 percent. The coefficient of variation of the K-5 method for the 3 x 6-in. (75 x 150 mm) cylinders was 36 percent. This high value is due to the very large number of variables used in compiling K-5 test data compared with the few variables in the ASTM round robin data. The coefficient of variation for the K-5 method would have been equivalent to about 18 percent if the tests had been performed on 6 x 12-in. (150 x 300-mm) cylinders, since large specimens have lower variance than small specimens under similar conditions. It is evident from the presented documentation that there is no significant difference in the accuracy of values determined by either the various accelerated test methods or the 28-day method. So, it is clear that accelerated testing of concrete is as good as the 28-day test that has been used for generations. However, accelerated testing has the advantage of giving the results in shorter time periods that vary between 5 and 48 hr after concrete is made, depending on the method used. Also, all concerned parties will be able to proceed with construction, take remedial action, or modify the mixture proportions in a timely manner. Early and fast action in the concrete industry, which should result from accelerated testing, will undoubtedly provide considerable savings, and reduce costly litigation to all concerned.

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* Personal communication from the late Robert E. Philleo.

Conclusions

From the discussion of the preceding data, the following conclusions can be made:

1. Prediction of 28-day strength based on the K-5 test method (ASTM C 684 Method D) is as good as that based on ASTM C 684 Method B (boiling-water method).

2. Accelerated test method K-5 (ASTM C 684 Method D) can play a very useful role in the acceptance and the control of the quality of concrete in the field and in ready-mixed as well as in precast plants.

PIN PENETRATION RESISTANCE OF HARDENED CONCRETE

Nondestructive testing of hardened concrete is used either to evaluate the required minimum safe strength for removal of concrete forms, or as a stand-alone tool to indicate the concrete quality, uniformity, and strength in structures.

There are several devices that are available for nondestructive testing of concrete and they include the hardness rebound hammer, the penetration probe, pullout, and ultrasonic tests (12). One of the new testers that became available recently is the pin penetration which conforms to ASTM C803 specification.

Description of the Apparatus

The apparatus which is shown in Fig. 15 can grip a pin (10 in Fig. 15) with a length of 1.2 in. (30.5 mm), a diameter of 0.14 in. (3.56 mm), and a tip machined at an angle of 22.5 deg. The pin is held by a hammer-shaped shaft (11) that is encased within the body (7) of the tester. the hammer is activated when it is tightened against the spring (12) by rotating and forcing its handle through the loading bolt (1). The spring stiffness is 284 lb/in. (49.7 N/mm) and the apparatus is built to store about 91 lb-in. (108 N-m) of energy.

When the hammer is loaded, the spring is compressed about 0.80 in. (20.3 mm), then released by the trigger (15) connected through the lever and catch (6).

In addition to the main apparatus, an air blower and a dial gage are required to measure the depth of penetration of the pin into the concrete. The complete apparatus and its accessories are shown in Fig. 16.

TEST PROGRAM

The tests were performed on concrete made with Type I portland cement and two types of aggregate: Mixtures A and B contained normal weight aggregate of glacial fluvial deposits from the Saskatoon area, while Mixtures C and D contained lightweight aggregate made from expanded clay and shale. A vinsol resin air-entraining agent was used in all mixtures. Details of the mixtures are given in Table 14.

Six 6 x 12 in. (150 x 300 m), twelve 3 x 6 in. (75 x 150 mm) cylinders, and one 18 x 24 x 4 in. (460 x 610 x 100 mm) slab were cast from each mixture. The cylinders were rodded 25 times in three layers, while the slabs were rodded in two layers, one rodding for every 2 in.^{2} (129 mm^2) of surface area. All specimens were covered with plastic sheets so as to minimize moisture loss. The next day, the specimens were stripped and cured under standard conditions and equivalent maturities. The 6x 12 in. (150 x 300 mm) cylinders were cured in a water bath in the humidity room while the 3 x 6 in. (75 x 150 mm) cylinders and the 18 x 24 x 4 in. (460 x 610 x 100 mm) slabs were cured in the same humidity room where the temperature and humidity were maintained respectively at 70 ± 3 F (21 ± 2 C) and 95 ± 3 percent. The 3 x 6 in. cylinders were used to estimate the strength of the slab when it was tested at different ages.

TEST RESULTS AND DISCUSSION

Strength tests were carried out for each mixture at ages of 1,2,5,7,14 and 28 days, and the results are plotted in Fig.17 where the ultimate compressive strength for normal and lightweight concrete was plotted against the pin penetration. Regression analysis was carried out and a linear relationship of the forms Y = A + BX was found satisfactory.

Additional tests were performed by Carino and Tank (13) and the results indicated that the precision for testing with the pin penetration apparatus can be summarized as shown in Table 15. This indexes of precision in Table 15 apply to measurements obtained by a single operator using the same instrument on the same concrete.

Conclusions

The following conclusions can be made from the above results:

1. A linear relationship exists between strengths and pin penetration for both normal and lightweight concrete.
2. The maximum range of 6 individual measurements for a single-operator is 0.064 in. (1.6 mm).
3. The maximum difference between two tests by a single-operator (each test calculated as the average of 6 measurements) is 0.018 in. (0.5 mm).
4. The new nondestructive pin penetration can be used to indicate the quality, uniformity, and strength of concrete in place.

5. It can be used to determine when it is safe to remove concrete forms or start prestressing operations.
6. The pin penetration apparatus is easy and inexpensive to use.

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Table 1. Statistical Analysis of Tests Performed at Laboratory B.

Type of Test	No. of Tests	Mean	Standard Deviation
Slump, in.	36	3.83	0.37
K-Slump, cm	36	3.55	0.90
Workability Index, cm	35	3.06	0.67

Table 2 - Mixture Proportions of Concrete

	lb	kg	
Cement	20	9.1	
Fine aggregate	64	2.9	
Coarse aggregate	76	3.5	
Air-entraining agent	22 cc		
	Water		
Material	lb	kg	w/c ratio
A	10.0	4.5	0.50
B	11.2	5.1	0.56
C	12.4	5.6	0.62
D	13.4	6.1	0.67

A/C = 7.0; CA/FA = 1.19

Table 3 Summary data sheet for inter-operator test program for the K-slump tester (all measurements of K-slump are in cm)

Operator number	Replicate measurements	Material			
		A	B	C	D
1	a	5	35	65	73
	b	10	35	65	73
	c	10	43	65	75
2	a	20	35	50	75
	b	8	35	50	63
	c	5	43	63	70
3	a	15	48	68	78
	b	8	48	60	68
	c	13	48	65	75
4	a	3	48	60	78
	b	8	40	63	75
	c	15	48	65	78

Table 4 - Precision* for single operator (all measurements of K-slump are in mm)

Average consistency	(1S) Limit[α]	Maximum range of three individual measurement[β]	(D2S) Limit[ς]	Maximum difference between two averages of three measurements**
2	6	19	16	9
4	5	17	14	8
6	4	14	12	7
8	4	13	11	6
Mean		16		7

* These values represent indexes of precision as described in recommended practice ASTM C 670.

[α] These values are the single-operator standard deviations for concrete made with the average consistency shown in Column 1.

[β] These values are the maximum allowable ranges for a group of three individual measurements made on the same concrete. If the range of three measurements exceeds the limit given, the three should not be averaged. In this case, a fourth measurement should be taken, and the one that deviates the most from the four measurements should be discarded. If the three remaining measurements still do not meet the limit given, three new measurements should be obtained from a new sample.

[ς] These values are the maximum difference between two individual measurements [(D2S) limits]. These indexes should not be used if measurements are being obtained in groups of three and averaged.

** A difference larger than the values given indicates the probability that there is a significant difference in the concrete being tested and represented by the two groups of three measurements each.

Table 5 -Precision* for multioperators
(all measurements of K-slump are in mm)

Average consistency	(1S) Limit$^{\alpha}$	Maximum range of three individual measurement$^{\beta}$	(D2S) Limit$^{\varsigma}$	Maximum difference between two averages of three measurements**
2	5	16	14	8
4	6	22	18	10
6	6	20	17	10
8	4	15	13	7
Mean		18		9

* These values represent indexes of precision as described in recommended practice ASTM C 670.

α These values are the multioperators standard deviations for concrete made with the average consistency shown in Column 1.

β These values are the maximum allowable ranges for a group of three individual measurements made on the same concrete. If the range of three measurements exceeds the limit given, the three should not be averaged. In this case, a fourth measurement should be taken, and the one that deviates the most from the four measurements should be discarded. If the three remaining measurements still do not meet the limit given, three new measurements should be obtained from a new sample.

ς These values are the maximum defference between two individual measurements [(D2S) limits]. These indexes should not be used if measurements are being obtained in groups of three and averaged.

** A difference larger than the values given indicates the probability that there is a significant difference in the concrete being tested and represented by the two groups of three measurements each.

Table 6 - Mix Proportions

	High Strength NWA			Medium Strength NWA				Low Strength NWA						Medium Strength NWA				Medium Strength LWA			
Concrete	**A**	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**I**	**J**	**K**	**L**	**M**	**N**	**O**	**P**	**Q**	**U**	**R**	**S**	**T**
Cement, pcy	838	850	848	645	650	657	652	450	450	445	460	426	476	653	657	658	661	910	901	888	881
Silica fume, pcy	59	60	59	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
W/CM	0.23	0.23	0.23	0.34	0.34	0.34	0.34	0.50	0.50	0.50	0.50	0.51	0.53	0.34	0.34	0.34	0.34	0.30	0.30	0.30	0.30
Sand, pcy	1194	1176	1139	1396	1350	1326	1278	1546	1485	1428	1440	1405	1572	1406	1362	1326	1293	1443	1430	1408	1397
Coarse agg., pcy	1891	1864	1805	1875	1813	1780	1715	1838	1766	1700	1712	1670	1869	1888	1829	1780	1736	719	711	701	696
MSA, in.	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	0.38	1.00	1.00	1.00	1.00	0.38	0.38	0.38	0.38
H-RWR, oz/cyd	377	372	363	194	130	131	130	135	90	89	92	170	238	193	131	132	119	173	171	169	167
AEA oz/cyd	0	4.5	13.6	0	4.9	8.2	10.4	0	3.4	5.6	4.1	0	0	0	4.9	8.2	9.9	0	4.5	7.1	11
AEA, pcy	0	0	0	0	0	0	0	0	0	0	0	1.5	1.7	--	--	--	--	--	--	--	--
Air-detraining, pcy	0	0	0	0	0	0	0	0	0	0	0	0	0.7	--	--	--	--	--	--	--	--

Note: cyd = yd^3; pcy - lb/yd^3

W/CM = water-cementitious material ratio

AEA = air entraining material ratio

MSA = maximum size aggregate

NWA = normal-weight aggregate

H-RWR = high-range water reducer

AWA = antiwashout admixture

LWA = lightweight aggregate

1 pcy = 0.593 km/m^3

1 oz = 29.57 ml

1 lb = 0.454 kg

Table 7 - Test Results

Concrete	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	U	R	S	T
K-Slump	3.00	5.00	8.00	1.25	1.50	2.00	4.00	2.50	1.50	3.50	4.75	1.25	1.50	3.50	9.25	11.00	7.00	2.00	3.00	2.00	4.50
Fresh unit weight, pcf	155.5	154.7	150.9	153.4	149.5	147.8	143.3	150.5	145.5	140.5	142.3	138.1	--	154.6	150.9	147.8	145.0	123.9	122.8	121.0	120.0
Air Volume of Fresh Concrete, %																					
Gravimetric	0.60	1.00	3.30	1.40	3.90	4.80	7.60	2.30	5.50	8.60	7.20	9.90	--	0.6	3.0	4.8	6.5	3.1	4.0	5.4	6.1
Pressure	2.00	2.50	4.75	2.00	4.25	6.25	8.50	2.00	5.00	11.75	8.75	12.50	4.25	1.25	4.20	6.00	9.00	--	--	--	--
Roller	1.75	2.25	4.50	1.75	4.00	5.75	8.25	2.50	5.25	8.50	7.25	10.25	4.25	1.50	3.75	5.50	7.50	2.50	3.75	5.25	8.25
Mini	2.00	2.50	4.75	2.00	4.75	6.25	8.50	2.50	4.75	9.50	7.00	10.50	4.00	1.50	4.00	5.50	8.00	2.50	4.25	6.00	8.50
Air Volume of Hardened Concrete, %																					
Linear traverse	2.0	2.1	3.6	1.7	3.7	5.1	6.8	2.5	4.5	6.4	5.5	9.0	3.8	1.4	4.0	5.2	6.3	2.2	3.1	4.4	7.0
28-day, f'c.ksi																					
No. 1	12.71	12.10	11.66	--	8.51	8.01	7.65	--	5.08	4.98	5.25	4.13	5.44	--	7.62	7.40	7.42	7.18	6.20	6.26	5.21
No. 2	12.29	11.90	11.55	--	8.35	7.87	7.56	--	4.70	4.65	4.97	4.07	5.44	--	7.12	7.01	7.04	6.62	6.02	6.26	5.15
Mean	12.50	12.00	11.61	--	8.43	7.94	7.61	--	4.89	4.82	5.11	4.10	5.44	--	7.37	7.21	7.23	6.90	6.11	6.26	5.18

Note: pcf = lb/ft^3; 1 pcf = 16.0 kg/m^3; 1 ksi = 6.897 MPa

Table 8 - Mixture Proportion Data and Properties of Fresh Concrete

Mixture	Mixture proportion			Properties of fresh concrete		
	w/c, by weight	A/C	Cement content kg/m^3	Unit weight, kg/m^3*	Air content, percent	Slump, mm**
1A	0.70	8.0	246	2395	2.2	90
1B	0.70	8.3	240	2395	2.4	75
2	0.65	7.5	262	2415	2.1	75
3A	0.60	7.0	279	2383	2.2	90
3B	0.60	7.0	279	2383	2.2	100
4	0.55	6.4	301	2395	2.0	90
5A	0.50	5.9	325	2408	2.0	70
5B	0.50	5.9	325	2408	2.0	75
6	0.45	5.2	362	2421	1.9	75
7A	0.40	4.4	415	2421	2.0	75
7B	0.40	4.4	415	2421	1.8	65
8	0.36	3.7	482	2440	1.8	75
9	0.70	8.8	209	2203	6.5	100
10	0.65	8.0	245	2363	4.5	65
11	0.60	7.5	251	2280	6.0	130
12	0.55	7.1	267	2325	4.5	75
13	0.50	6.4	298	2357	4.9	75
14	0.45	5.7	333	2370	4.9	45
15	0.40	4.3	420	2376	4.5	75
16	0.36	3.6	483	2383	4.9	65

* 1 kg/m^3 = 0.06243 lb/ft^3

** 1 mm = 0.03937 in.

Table 9 - Summary of Regression Equations in

Fig. 10 and 11

Type of concrete	Age at test	Regression Equations MPa	Coefficient of correlation "r"
Non-A.E.A.*	7 days	$Y = 7.97 + 1.19\ X$	0.96
A.E.A.		$Y = 8.00 + 0.91\ X$	0.99
Non-A.E.A.	28 days	$Y = 11.33 + 1.40\ X$	0.97
A.E.A.		$Y = 10.84 + 1.04\ X$	0.99

*A.E.A. = air-entraining admixture

Table 10 - ASTM Round Robin Mixture Proportion Data and Properties of Fresh Concrete

Mix Proportion				Properties of Fresh Concrete		
Mix No.	w/c, + F* by weight	Aggregate/C + F, by weight	C + F content, kg/m^3	Unit weight, kg/m^3**	Air content, percent	Slump, mm
C	0.70	9.5	201	2242	6.5	50
F1	0.70	9.4	208	2312	6.2	40
F2	0.70	9.4	197	2196	8.0	90
F3	0.60	9.5	201	2242	6.6	85
F4	0.70	9.5	200	2228	7.2	70
F5	0.70	9.4	197	2190	8.5	55
F6	0.70	9.4	199	2216	6.9	90
F7	0.70	9.4	202	2254	7.0	100
F8	0.70	9.4	202	2254	6.8	70
F9	0.70	9.4	202	2342	7.0	90
C	0.50	6.4	295	2331	6.3	75
F1	0.50	6.4	294	2318	6.1	100
F2	0.50	6.4	292	2312	6.2	65
F3	0.50	6.4	292	2306	6.6	95
F4	0.50	6.4	289	2291	7.0	50
F5	0.50	6.4	289	2293	7.0	55
F6	0.50	6.4	289	2293	6.9	115
F7	0.50	6.4	288	2280	7.0	100
F8	0.50	6.4	295	2331	6.0	75
F9	0.50	6.4	294	2318	6.5	90
C	0.40	4.3	412	2331	6.3	75
F1	0.40	4.3	404	2286	6.8	135
F2	0.40	4.3	407	2306	6.1	110
F3	0.40	4.3	404	2286	6.6	150
F4	0.40	4.3	408	2312	6.1	90
F5	0.40	4.3	405	2293	7.2	70
F6	0.40	4.3	408	2312	6.5	180
F7	0.40	4.3	407	2306	6.9	160
F8	0.40	4.3	400	2267	7.3	180
F9	0.40	4.3	407	2306	6.2	150

* Each mix had fly ash from a different source. It was 25 percent by weight of cement.

** 1 kg/m^3 = 0.06243 lb/ft^3.

Table 11 - Summary of Accelerated and Compressive Strength Results for the K-5 and the ASTM Round Robin Test Data

w/c	No.	Accelerated Strength				Compressive Strength			
		K-5 method		ASTM Procedure B		7 day		28 day	
		psi	MPa	psi	MPa	psi	MPa	psi	MPa
0.70	C	1100	7.6	1310	9.0	2120	14.6	2660	18.3
	F1	1130	7.8	–	–	1690	11.7	2350	16.2
	F2	800	5.5	630	5.7	1300	9.0	1840	12.7
	F4	930	6.4	1030	7.1	1630	11.2	2150	14.8
	F5	830	5.7	1030	7.1	1490	10.3	2100	14.5
	F6	600	4.1	790	5.4	1480	10.2	2000	13.8
	F7	600	4.1	720	5.0	1360	9.4	1790	12.3
	F8	770	5.3	970	6.7	1560	10.8	2120	14.6
	F9	730	5.0	890	6.1	1600	11.0	2210	15.2
0.50	C	2530	17.4	2380	16.4	3360	23.2	4050	27.9
	F1	1870	12.9	2170	15.0	2910	20.1	3940	27.2
	F2	2270	15.7	2310	15.9	3030	20.9	4090	28.2
	F3	2230	15.4	2610	18.0	2940	20.3	4230	29.2
	F4	1630	11.2	1810	12.5	2680	18.5	3590	24.8
	F5	2030	14.0	2190	15.1	2920	20.1	3430	27.1
	F6	1330	9.2	1590	11.0	2660	18.3	3400	23.4
	F7	1470	10.1	1700	11.7	2820	19.4	3540	14.4
	F8	2130	14.7	2300	15.9	2930	20.2	3900	26.9
	F9	1770	12.2	1960	13.5	2870	19.8	3840	26.5
0.40	C	3730	25.7	2950	20.3	3990	27.5	4850	33.4
	F1	2670	18.4	2660	18.3	3200	22.1	4080	28.1
	F2	2800	19.3	2810	19.4	[illegible]	25.5	4610	31.8
	F3	2400	16.5	2980	20.5	[illegible]	22.9	4360	30.1
	F4	3200	22.1	2930	20.2	[illegible]	24.6	4480	30.9
	F5	2530	17.4	2900	20.0	3460	23.9	4360	30.1
	F6	2400	16.5	2340	16.1	3340	23.0	4190	28.9
	F7	2430	16.8	2470	17.0	3430	23.6	4290	29.6
	F8	2500	17.2	2540	17.5	3160	21.8	4220	29.1
	F9	2600	17.9	2810	19.4	3610	24.9	2490	31.0

Compressive strength and ASTM Procedure B results are the average of two 150 x 300 mm (6 x 12 in.) specimens. K-5 method results are the average of three 75 x 150 mm (3 x 6 in.) specimens.

Table 12 - Summary of Regression Equations in Fig. 12 and 13.

Procedure	Age at test	Regression Equations, MPa	Coefficient of correlation "r"
ASTM C 684 Procedure B	7 days 28 days	$Y = 4.72 + 1.02\,X$ $Y = 6.53 + 1.27\,X$	0.97 0.96
K-5 method	7 days 28 days	$Y = 6.68 + 0.93\,X$ $Y = 9.30 + 1.13\,X$	0.96 0.94
K5 ASTM C 684 Procedure B		$Y = 2.50 + 0.87\,X$	0.96

Table 13 - Summary of the Ratios of the Accelerated to the 91-day Strength of Concrete

		Method A 24 hr - 35 C (95 F)		Method B C 23 hr - 21 C (70 F) $\frac{1}{2}$ hr - boiling				Method C autogenous 46 hr		Combined A,B,C	K-5 method
		$\frac{a}{91} = \frac{\text{Accelerated strength}}{\text{91-day strength}}$									
Cement content kg/m^3	450	550	650	450	550	650	450	550	650		
n	7	9	9	5	6	7	6	6	6	61	337
Ave	.376	.417	.430	.443	.439	.474	.529	.577	.595	.476	.431
S	.080	.057	.088	.101	.066	.085	.106	.090	.100	.087	.153
CV	21	14	21	23	15	18	20	16	17	18	36*

n = number of tests; *Ave* = average of accelerated strength over 91-day strength; S = variance; *CV* = coefficient of variation.

* This high value is due both to the small size of the cylinders used, 3 x 6 in. (75 x 150 mm) and to the very large number of variables used in compiling the K-5 test data compared with only the few variables in the ASTM round robin data.

Table 14 - Summary of Mixtures and Control Test Results

Materials, weight per batch	Mix A		Mix B		Mix C		Mix D	
	lb	kg	lb	kg	lb	kg	lb	kg
Water	25.8	(11.7)	23.9	10.8	30.8	13.9	28.0	12.7
Cement	43.0	(19.5)	45.0	20.4	47.7	21.6	50.0	22.7
Coarse aggregate	165.5	(75.1)	173.0	78.5	72.7	33.0	77.5	35.2
Fine aggregate	135.5	(61.5)	142.0	64.5	96.3	43.7	102.5	46.5
Ratios by weight:								
Water-Cement	0.6		0.53		0.66		0.56	
Aggregate Cement	7.0		7.0		3.6		3.6	
Coarse-aggregate-fine aggregate	1.22		1.22		0.76		0.76	
Aggregate type	Normal		Normal		Lightweight		Lightweight	
Air-entraining agent	30cc		30cc		25cc		25cc	
Air content, percent	4.5		5.4		9.6		9.6	
K-slump,cm	4.5		2.0		4.0		3.0	
K-workability, cm	4.5		3.0		2.75		2.5	

Table 15 - Precision[A] Resistance Testing with Pins

Material	(1S) Limit[B], in. (mm)	Maximum Range of Six Individual[C] Measurements in. (mm)	(D2S) Limit[D] Maximum Difference Between TwoTests (Each test calculated as the average of six measurements),in, (mm)
Concrete 450-4000 psi (3.1-27.6 MPa)	0.016 (0.4)	0.064 (1.6)	0.018 (0.5)

[A] These values represent indexes of precision as described in Practice C 670.

[B] These values are the single-operator standard deviations for tests on concrete shown in Column 1.

[C] These values are the maximum allowable ranges for groups of six individual measurements made in a 12 in. (300 mm) diameter area. If the range of the six measurements exceeds the limit given, the six should not be averaged. In this case, a seventh pin should be fired, and the one that deviates the most from the average of the seven measurements should be discarded. If the six remaining measurements still do not meet the limit given, the device should be moved to a different area and six new measurements obtained.

[D] A difference larger than the values given indicates a high probability that there is a statistically significant difference in the concrete in the two areas represented by the two groups of six measurements each.

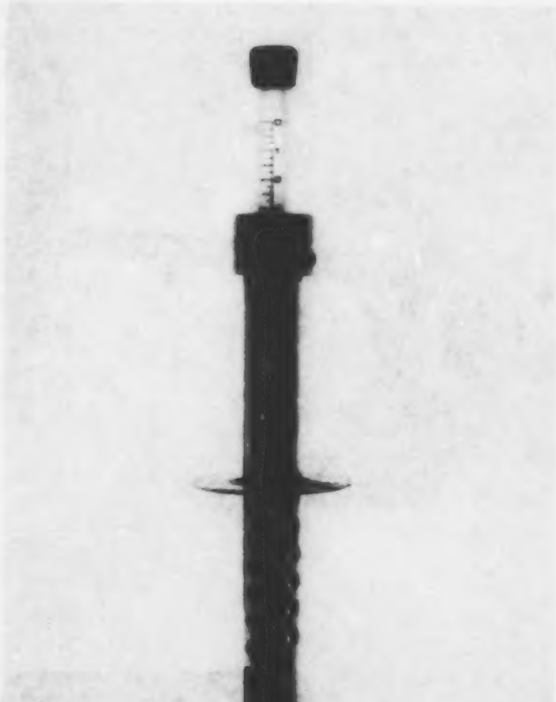


Fig. 1 Photograph of the K-Slump Tester

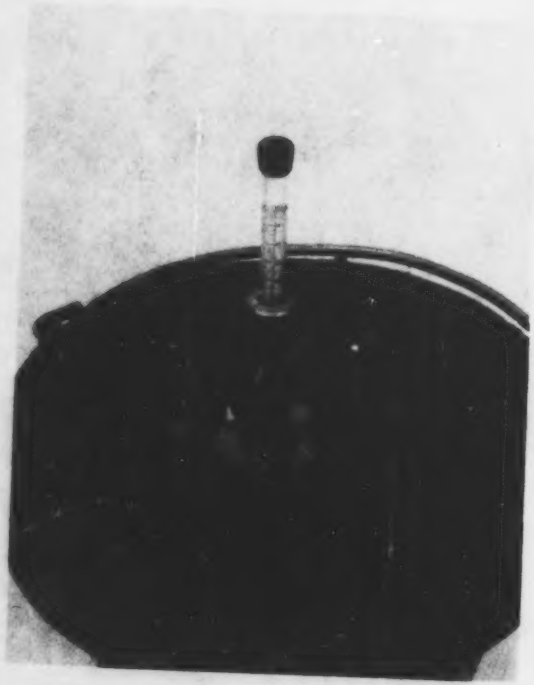


Fig. 2 Measuring the K-Slump

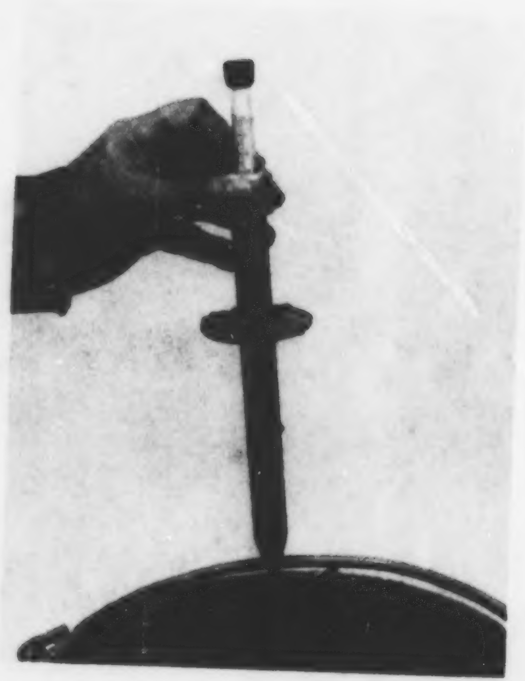


Fig. 3 Measuring the Workability

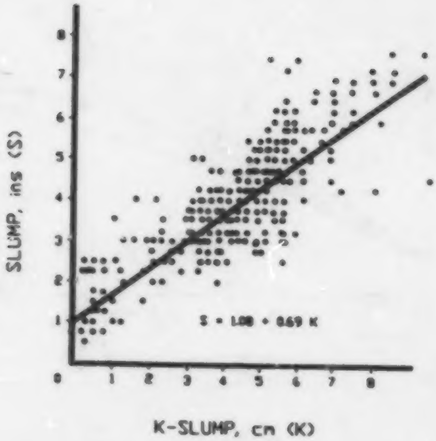


Fig. 4 Relation of Slump to K-Slump for the Data from Laboratories A, C, D, and E

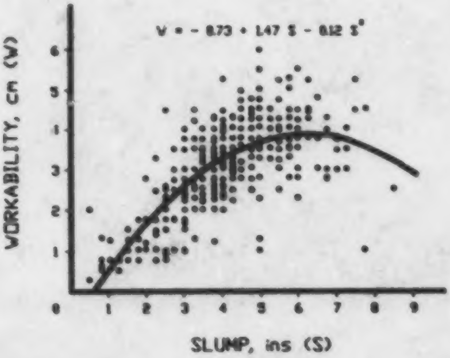


Fig. 5 Relation of Workability to Slump for the Data from Laboratories A, B, C, and E

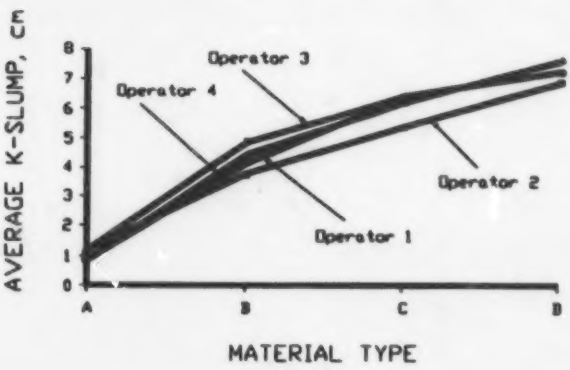


Fig. 6 Average K-Slump versus Material Type

Fig. 7 Photograph of the Pressure, Roller, and Mini Airmeters

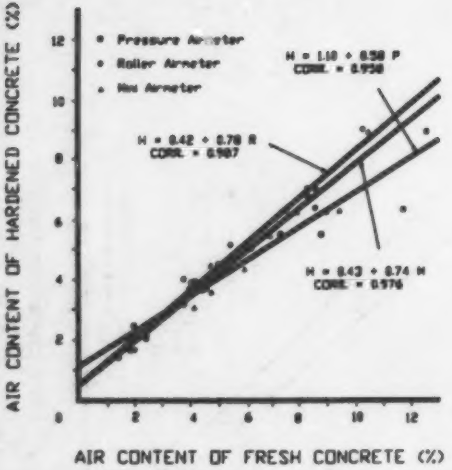


Fig. 8 Hardened Concrete Air Content versus Fresh Concrete Air Content

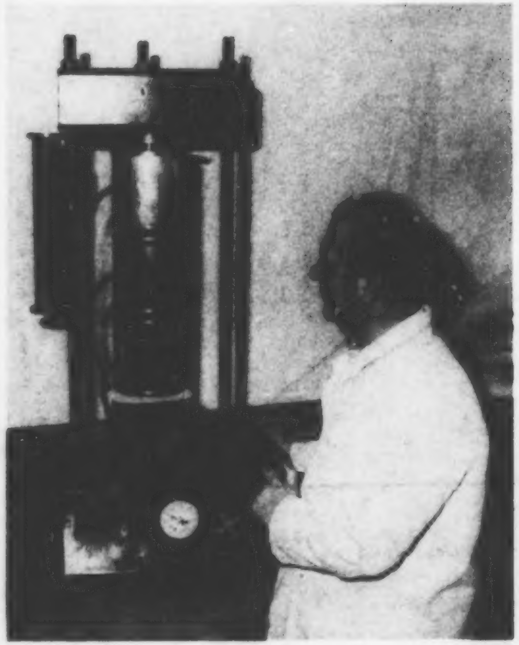


Fig. 9 K-5 Accelerated Strength Tester

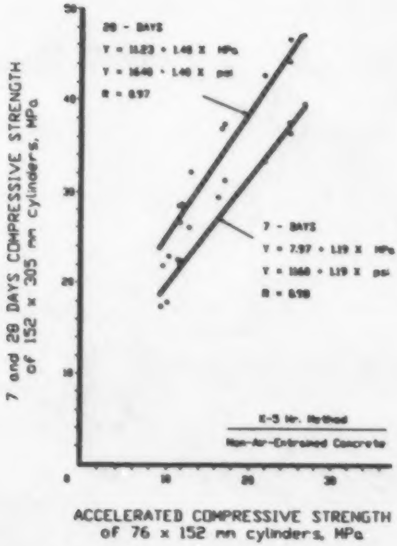


Fig. 10 Relation of 7 and 28 Day Strengths to the K-5 Accelerated Strength of Non-Air Entrained Concrete

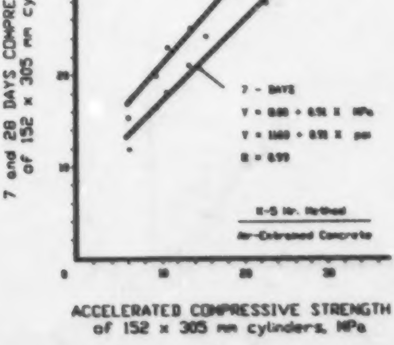


Fig. 11 Relation of 7 and 28 Day Strengths to the K-5 Accelerated Strength of Air Entrained Concrete

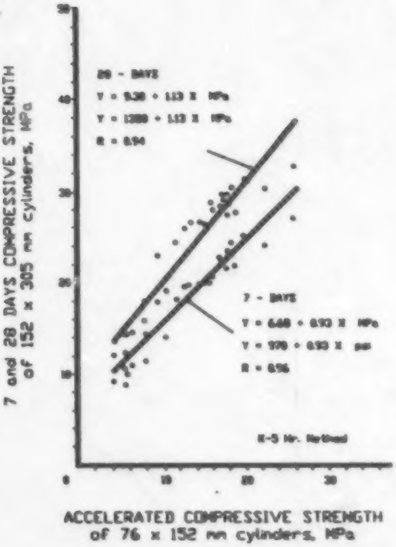


Fig. 12 Relation of 7 and 28 Day Strengths to the K-5 Accelerated Strength

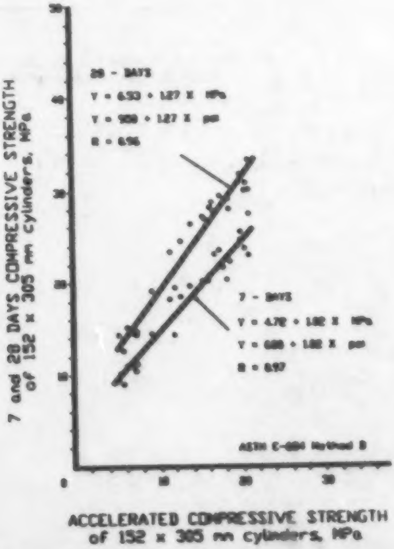


Fig. 13 Relation of 7 and 28 Day Strength to the ASTM C 684 Method B (boiling water) Accelerated Strength

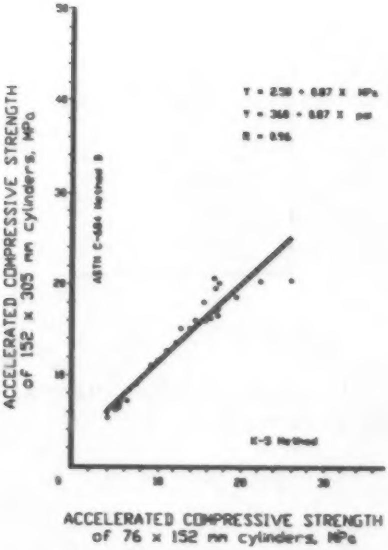
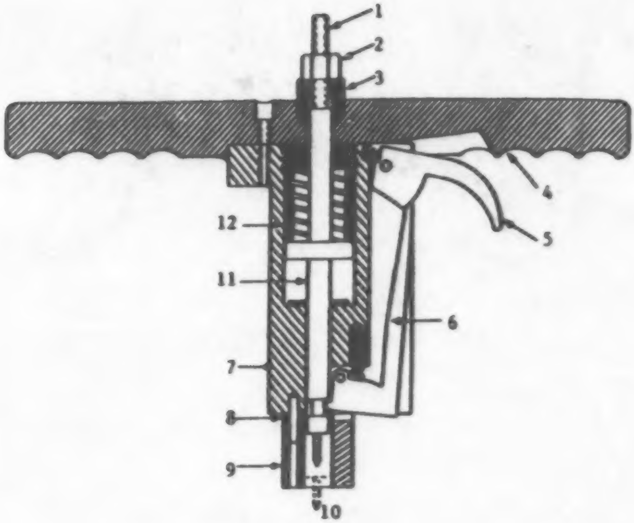


Fig. 14 Relation of ASTM Method B (boiling water) Accelerated Strength to the K-5 Accelerated Strength



1. Loading Bolt
2. Holding Nut
3. Holding Nut Support
4. Gripping Handle
5. Trigger
6. Trigger Lever and Catch
7. Main Frame
8. Depth Spacer
9. Spacer
10. Pin
11. Shaft and Hammer
12. Spring

Fig. 15 Descriptive Diagram of Nondestructive Pin Penetration Strength-Testing Apparatus.

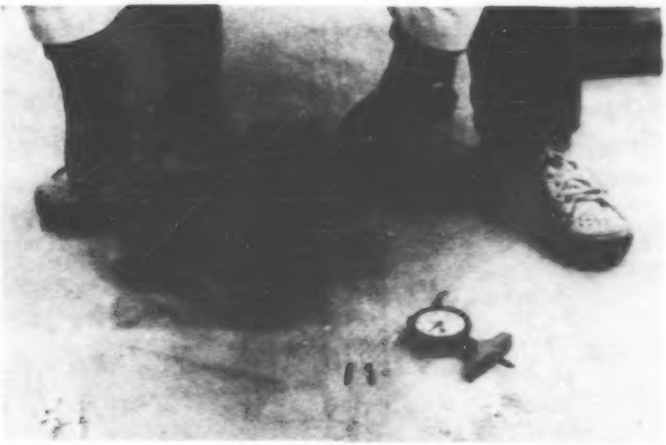


Fig. 16 Pin Penetration Test.

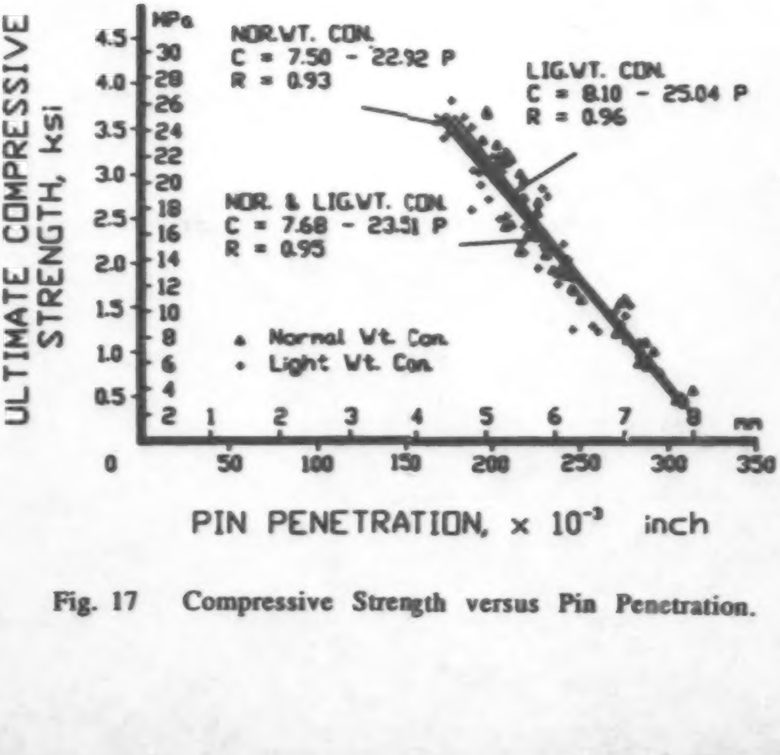


Fig. 17 Compressive Strength versus Pin Penetration.

Recent Developments in Nondestructive Testing of Concrete[1]

by Nicholas J. Carino

Synopsis: Advances in nondestructive testing of concrete are summarized. Test methods are classified into those used to assess in-place strength and those used to locate hidden defects. In the first category, recent developments are presented on the pullout test, the break-off test, the torque test, the pull-off test and the maturity method. In the second category, a review is presented of infrared thermography, ground penetrating radar, and several methods based upon stress wave propagation. The principles of the methods, their advantages and their inherent limitations are discussed. Where appropriate, requirements of relevant ASTM standards are discussed. Future directions for research in the area of nondestructive testing of concrete are presented.

Keywords: Break-off test; concrete; flaw detection; impact-echo; impulse response; impulse radar; infrared thermography; in-place strength; maturity; nondestructive testing; pull-off test; pullout test; short pulse radar; spectral analysis; stress wave propagation; surface waves; torque test; voids.

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[1] Prepared by employee of the U.S. government and not subject to copyright.

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INTRODUCTION

During the 1970s and 1980s there were dramatic increases in the amount of research on nondestructive testing (NDT) of concrete. In addition, practitioners have become more aware of the importance of NDT methods. Within the last five years, national and international conferences and symposia were held to summarize current knowledge and identify research needs. Possible causes for this surge in interest include: (1) the increase in the pace of concrete construction leading to the need for methods to assess in-place strength for safety; (2) a growing interest in the rehabilitation of existing structures, which requires methods for condition assessment; (3) and the advances made in computer hardware and digital signal analysis methods. However, there are still many problems that need to be solved to bring the nondestructive testing of concrete on par with the testing of metals.

The term "nondestructive testing" has been used loosely when applied to concrete. In general, NDT methods can be divided into two groups: (1) those which are used to estimate the in-place mechanical properties and (2) those which are used to locate hidden anomalies.

The earliest NDT methods were developed to estimate in-place strength, and many have been incorporated into standards. Some of these methods, such as rebound hammer and pulse velocity, are truly nondestructive, while others result in varying degrees of surface damage, such as the pullout or the break-off tests. In North America, the primary use of these strength methods has been for estimating in-place strength development during construction [1]. In-depth reviews of the historical development and the inherent advantages and limitations of these methods have been published [12, 54, 56]. More recently, there has been a growing interest in methods to evaluate the in-place permeability of the "covercrete", since this is recognized as the key parameter for durability of concrete. These methods are not covered in this review.

The second category of NDT methods are useful for condition assessment of existing structures. They rely on the interactions of internal discontinuities (cracks, voids, honeycombing, etc.) with the propagation of electromagnetic or stress waves. These methods include infrared thermography, ground penetrating radar, and impact-echo. In general, methods for flaw detection are not in an advanced stage of development and there is only limited standardization.

The purpose of this paper is to review some of the recent advances in nondestructive testing of concrete. Both categories of methods will be covered, but emphasis will be placed on those aspects in which the author has had personal involvement. An extensive reference list is provided to assist the reader interested in additional information.

STRENGTH METHODS

A variety of methods have been proposed over the years to estimate the in-place compressive strength of concrete. These are indirect methods which measure a property other than compressive strength, and they use a correlation relationship between the measured property and compressive strength to estimate the in-place strength. The ideal in-place test should have a correlation relationship with a narrow confidence band, which is applicable to any concrete mixture. Recent research studies have focused on understanding those factors which affect the correlation relationships and defining the statistical characteristics of the tests. The report by American Concrete Institute Committee 228 [1] discusses the statistical characteristics of the in-place test methods which have been standardized by the American Society for Testing and Materials (ASTM). The following discusses the recent advances in selected methods for estimating in-place strength.

Pullout Test

The pullout test is one of the most reliable techniques for estimating in-place strength of concrete during construction. In this method, an insert with an enlarged head is cast in the concrete. The insert and the accompanying conical fragment of concrete are extracted by using a tension loading device reacting against a bearing ring which is concentric with the insert. The force required to pull out the insert is an indicator of concrete strength. A discussion of the historical development and applications of the pullout test may be found in Chapter 3 of Malhotra and Carino [56].

Failure mechanism -- Ever since the test was first described by Skramtajew [83], there has been an incomplete understanding of its failure mechanism. Skramtajew correctly noted that the tests subjects the concrete to a combination of tensile and shearing stresses. Kierkegaard-Hansen [43], the inventor of the widely used Lok-Test system, tried to relate the shape of the extracted conical fragment to the intact cones often observed at the ends of cylinders tested in compression. Malhotra and Carette [55] proposed that the pullout strength was related to the direct shear strength of concrete. Recent experimental and analytical studies have tried to gain a better understanding of the failure process during the pullout test [5, 38, 46, 47, 71, 87, 90].

From these independent analytical and experimental studies, it is now understood that the pullout test subjects the concrete to a nonuniform, three-dimensional state of stress. It also has been demonstrated that the failure process involves two circumferential crack systems: a stable primary system which initiates at the insert head at about 1/3 of the ultimate load and propagates into the concrete at a large apex angle; and a secondary system which propagates with increasing load and defines the shape of the extracted cone. Figure 1 shows these cracking systems as predicted by Hellier et al. [38] using a discrete cracking, finite-element model based on non-linear fracture mechanics. However, there is still no consensus on the failure mechanism at the ultimate load. Some believe that ultimate load occurs as a result of compressive failure along a "strut" running from the bottom of the bearing ring to the insert head [46, 47, 71]. This mechanism has been used to explain the good correlation between pullout strength and compressive strength. Others believe that failure is governed by aggregate interlock across the secondary crack system, and the ultimate load is reached when sufficient aggregate particles have been pulled out of the mortar matrix [38, 87]. In this case, it is argued that there is correlation between pullout strength and compressive strength because both properties are controlled by the tensile strength of the mortar. In the compression test, the ultimate load is associated with the formation and growth of microcracks through the mortar.

Despite the lack of agreement on the exact failure mechanism, it has been shown that the pullout strength has good correlation with the compressive strength of concrete and that the test has good repeatability. In a review of published data, ACI Committee 228 recommends a coefficient of variation (standard deviation divided by the mean) of 8% for the pullout test. Recent modifications of ASTM C 900 [4] require a minimum of five individual pullout tests for each 115 m^3 of concrete in a given placement.

Post-installed tests -- A drawback of the standard pullout test is that the locations of the inserts have to be planned in advance of concrete placement and the inserts have to be fastened to the formwork. This limits the applicability of the standard method to new construction. In an effort to extend the application of pullout testing to existing structures, various techniques have been investigated for performing "post-installed" pullout tests. Some of the more promising approaches are shown in Fig. 2. However, none of these methods have been standardized by ASTM.

During the 1970s, a need arose in the United Kingdom for in-place tests to evaluate distressed concrete structures that had been built using high alumina cement. Chabowski and Bryden-Smith [23, 24] of the Building Research Establishment (BRE) developed a pullout technique based on commercial anchor bolts with expanding sleeves, as shown in Fig. 2(a). A 6-mm diameter hole is drilled into the concrete and an anchor bolt is inserted into the hole so that the split-sleeve is at a depth of 20 mm. After applying an initial load to expand and engage the sleeve, the bolt is loaded in tension and the maximum load during the extraction is recorded. Reaction is provided by three "feet" located along the perimeter of a 80-mm diameter ring. The sleeve transfers vertical and horizontal load components to the concrete. The fracture surface differs from that in the standard pullout test, and the test has been called an "internal fracture test" rather than a "pullout test". The correlation relationship between ultimate load and compressive strength has been found to have a pronounced non-linearity, indicating that the failure mechanism is probably related to the tensile strength of the concrete. Within-test variability was found to be greater than the standard pullout test, and the 95% confidence limits of the correlation relationship were found to range between ±30% of the mean curve [23].

In the BRE test system, the pullout force is applied by turning a nut on the end of the anchor bolt and measuring the maximum torque achieved during the test. Bungey [11] developed a mechanical loading system with the aim of reducing the scatter of test results compared with the torque loading system. Using the mechanical loading system, the 95% confidence intervals for the estimated compressive strength were estimated to be ±20%, which is a significant improvement. The low precision of the internal fracture test has been attributed to two principal causes [11]: (1) the variability in the hole drilling and preparation; and (2) the influence of aggregate particles on the load transfer mechanism and the failure initiation load.

Mailhot, et al. [53] investigated the feasibility of several post-installed pullout tests. One of these used a split-sleeve and a tapered bolt. In this case, the bolt assembly was placed in a 19-mm diameter hole drilled into the concrete. As shown in Fig. 2(b), the details differ from the BRE method because the reaction to the pulling force acts through a specially designed high strength, split-sleeve assembly. Thus the force transmitted to the concrete is predominantly a lateral load due to the expansion of the sleeve. It is claimed that failure occurs by shear, but it is more likely that failure occurs by splitting similar to that which occurs in a standard splitting tension test. As with the BRE test, the variability of this test was reported to be rather high. A second successful method involved epoxy-grouting a 16-mm diameter threaded rod to a depth of 38 mm in a 19-mm diameter hole. After the epoxy had cured, the rod was pulled using a tension jack reacting against a bearing ring. This method was also reported to have high variability. The study concluded that these two methods have the potential for assessing the strength in existing construction. However, additional research was recommended to enhance their reliability.

Domone and Castro [29] also developed a technique similar to the expanding sleeve method shown in Fig. 2(b). However, a torque meter was used to apply the load and the embedment was 20 mm as in the BRE method. Based on a limited number of tests it was concluded that this method gave better correlation relationships than the BRE method.

Another method was developed by the manufacturer [72] of the LOK-TEST system and is referred to as the CAPO test (for Cut And PullOut). The method involves drilling an 18-mm diameter hole into the concrete and using a special milling tool to undercut a 25-mm diameter slot at a depth of 25 mm. An expandable ring is placed in the hole, and the ring is expanded using special hardware. Figure 2(c) shows the ring after expansion. The entire assembly used to expand the ring is pulled out of the concrete using the same loading system as for an ordinary pullout test. Thus, unlike the other methods discussed above, the CAPO test subjects the concrete to a similar type of loading as the standard pullout test. The performance of the CAPO test in laboratory evaluations has been reported to be similar to the LOK-TEST [72]. Users of the CAPO test have indicated that the test is cumbersome to perform and care is needed to control the variability[2]. The test surface must be flat and perpendicular to the drilled hole. If these conditions are not achieved, the bearing ring will not seat properly and test results will be erratic.

Other Strength Methods

In addition to developments related to pullout testing, there have been other advances in methods for measuring in-place strength. One of these, the break-off test, was recently standardized by ASTM, and the pin penetration test was added to ASTM C 803 [4].

Break-off test -- The break-off test measures the force required to break off a cylindrical core from the concrete mass [41]. The measured force and a pre-established correlation relationship are used to estimate the in-place compressive strength. Standard procedures for using this method are given in ASTM C 1150 [4]. Naik presents a review of the research results in Chapter 4 of the handbook of Malhotra and Carino [56].

The principle of the break-off test is illustrated in Fig. 3(a). For new construction, the core is formed by inserting a plastic sleeve into the fresh concrete. When the in-place compressive strength is to be estimated, the sleeve is removed, and a special loading jack is placed into the counter bore. A hand-operated hydraulic pump is used to supply hydraulic fluid to the jack, which applies a horizontal force to the top of the core. The force on the core is gradually increased until the core ruptures at its base. The hydraulic fluid pressure is monitored with a pressure gage, and the maximum pressure gage reading in units of bars (1 bar = 0.1 MPa) is referred to as the "break-off number" of the concrete.

For new construction, the sleeves are inserted into the top surface of the member after the concrete has been leveled. Alternatively, the sleeves can be attached to the sides of the formwork and filled during concrete placement. For existing construction, a special drill bit can be used to cut the core and the counter-bore.

For ease of sleeve insertion into the fresh concrete, the concrete must be workable.

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[2] Read, P.H., Bickley, J.A., and Omran, R., "Simulated Field Trials," Draft Report for SHRP Contract 88-C204, January 1991

In addition, to minimize interference, the maximum aggregate size should be limited to a fraction of the sleeve diameter, which is 55 mm. According to ASTM C 1150, the break-off test is not recommended for concrete having a maximum nominal aggregate size greater than 25 mm. Sleeve insertion must be performed carefully to assure good compaction around the sleeve and a minimum of disturbance at the base of the formed core. Some problems have been reported in keeping the sleeves from "floating" out of very fluid concrete mixtures [61].

As is the case with the pullout test, the break-off test subjects the concrete to a slowly applied force and measures a static strength property of the concrete. In essence, the core is loaded as a cantilever, and the concrete at the base of the core is subject to a combination of bending and shearing stresses. In the early work [41], the results of the break-off test were reported as the "break-off strength," by computing the flexural stress at the base of the core corresponding to rupture force. In more recent applications of the method (e.g., ASTM C 1150), the flexural strength is not computed, and the break-off number (pressure gage reading) is related directly to the compressive strength. This approach simplifies data analysis, but it is mandatory to calibrate the test instruments used in the field to assure that the gage readings correspond to the actual forces applied to the test specimen.

The correlation relationships between break-off strength and compressive strength have been found to be non-linear [6, 41], which is in accordance with the usual practice of relating the modulus of rupture of concrete to the square root of compressive strength. It has also been found that the correlation relationship between break-off strength and modulus of rupture may be more uncertain than that between break-off strength and compressive strength [6].

Failure during the break-off test occurs by fracture at the base of the 55-mm diameter core. The crack initiates at the most highly stressed point on the core and propagates through the mortar and, in most cases, around coarse aggregate particles located at the base of the core. The ultimate load in each test would be influenced by the particular arrangement of aggregate particles within the failure region. Because of the relatively small size of the fracture surface and the heterogeneous nature of concrete, the distribution of aggregate particles will be different at each test location. Hence one would expect the within-test variability of the break-off test to be higher than that of other standard strength tests which involve larger test specimens. One would also expect that the variability might be affected by maximum aggregate size and aggregate shape.

The developer of the break-off test reported a within-test coefficient of variation of about 9% [41]. This value has generally been confirmed by other investigators. Table 1 summarizes some of the published data on within-test variability of the break-off test. The results have been grouped according to nominal maximum aggregate size and aggregate type (river gravel and crushed stone). The number of replicate tests used to determine the average and standard deviation are also listed. Where more than one data group was reported, the table lists the ranges and averages of the coefficients of variation. The following observations can be made:

- The variability tends to increase with increasing maximum aggregate size.
- The variability in concrete made with river gravel tends to be higher than in concrete made with crushed stone.

The variability reported by Barker and Ramirez [6] appears to be to be lower than that reported by others. Part of the reason for this difference may be because of the testing technique. In most of the research, break-off tests have been performed on slab specimens. However, Barker and Ramirez inserted the sleeves into the tops of a 152 x 152-mm cylinders. It is possible that the confining effects of the cylinder mold produced more reproducible conditions at the base of the cores.

The results of Naik et al. [61] suggest that the variability of break-off tests on drilled cores is comparable to that obtained on formed cores. However, cores were not drilled into concrete having compressive strength less than about 3000 psi (21 MPa). Thus additional data are needed to determine the lowest concrete strength for which core drilling does not affect the integrity of the concrete at the base of the core.

In summary, the results summarized in Table 1 support Johansen's [41] contention that the break-off test has a within-test coefficient of variation of about 9%. The variability is expected to be slightly higher for concrete with nominal maximum aggregate size greater then 19 mm. In ASTM C 1150, at least five individual break-off tests are required for each 100 m^3 of concrete in a given placement.

The break-off test has been used successfully on a variety of construction projects in the Nordic countries, including some of the major offshore oil platforms [22]. In addition to its use for estimating in-place compressive strength, the method has also been used to evaluate the bond strength between concrete and overlay materials [27].

Pin penetration test -- In the probe penetration test, covered by ASTM Test Method C 803 [4], a steel rod (probe) is driven into the concrete by a powder-actuated driver, and the penetration depth is correlated with compressive strength. The commercial test system to perform this test is known as the Windsor probe. Recently, a pin penetration test device, which imparts less energy than the Windsor probe system, was developed [62, 63]. The procedure for using the pin penetration test was incorporated into ASTM C 803. A spring-loaded device is used to drive a 3.6-mm (0.140-in.) diameter hardened steel pin into the concrete. The penetration by the pin creates a small indentation (or crater) on the surface of the concrete. The pin is removed from the crater, the crater is cleaned with an air jet, and the crater depth is measured with a suitable depth gage.

The kinetic energy delivered by the pin driver is estimated to be about 1.3% of the energy delivered by the Windsor probe system [19]. Because of the low energy level, the pin penetration is greatly reduced if the pin encounters a coarse aggregate particle. Thus the test is primarily a penetration test of the mortar fraction. Individual test results which penetrate coarse aggregates particle are not considered in determining the average pin penetration (see ASTM C 803). It has been found that the sensitivity of the pin penetration to changes in compressive strength decreases as the concrete strength increases above about 28 MPa (4000 psi) [19]. Therefore, the pin penetration test system is not recommended for testing concrete having a compressive strength above 28 MPa (4000 psi).

There are limited data on the repeatability of the pin penetration test. Nasser and Al-Manaseer [63] reported an average coefficient of variation of about 5% for replicate tests on 100-mm (4-in.) thick slab specimens and on the bottom surfaces of 152 x 305-mm (6 x 12-in.) cylinders. The variability was based on the "best five of seven readings," and the concrete strength varied from about 3.5 to 24 MPa (500 to 3500 psi). In another study [19], eight replicate pin tests were performed at the mid-height of 100 x 200-mm (4 x 8-in.) cylinders. The compressive strengths ranged from about 7 to 40 MPa (1000 to 5800 psi). Each set of replicate penetration depths was analyzed for potential outliers due to penetrations into large air voids or coarse aggregate particles. On average, two of the eight readings were discarded. The average standard deviation was found to be 0.41 mm (0.016 in.), and this value was adopted in the precision statement of ASTM C 803. Additional data are needed on the repeatability and the field performance of the pin penetration test.

Pull-off test -- This method was also developed as a result of the need in the United Kingdom to assess the in-place strength of distressed concrete structures made with alumina cement [52]. A 50-mm diameter metal disk is glued to the concrete surface using an epoxy adhesive. After the adhesive has cured the disk is pulled in tension, and a subsurface fracture occurs in the concrete (see Fig 3(b).) The nominal tensile strength is determined by dividing the maximum load by the area of the disk, and the compressive strength is estimated from a previously established correlation relationship.

The test can be performed with or without a partial core as shown in Fig. 3(b). The partial core is useful when the concrete has an outer hard shell, for example, as a result of carbonation. The partial core technique can also be used to assess the bond strength of overlay and repair materials to the base concrete. In this case, the core is drilled through the overlay and for a short distance into the base concrete. Surface laitance and organic materials (curing compound, form release agents, etc.) must be removed prior to gluing the disk to assure a strong bond between the disk and the core.

The pull-off test is essentially a direct tension test of concrete. Therefore, the relationship between pull-off strength and compressive strength is affected by those factors which affect the relationship between tensile and compressive strength. These would include the age of concrete, aggregate type and size, curing conditions, and air entrainment [52]. In comparative studies, it was reported that the pull-off test was more reliable than the BRE internal fracture test and slightly more reliable than the Windsor probe test (Long and Murray, 1984). The within-test coefficient of variation in laboratory studies has been reported to be in the range of 11 to 17 %, depending on the age of the concrete [52].

While most of the research has dealt with the use of the pull-off method to estimate the in-place compressive strength, it may have potential as an effective quality control tool for monitoring the effectiveness of on-site curing.

Stoll Tork Test -- This relatively new test method requires the embedment of an insert as is the case for the pullout test. However, the insert can be placed at any depth within the concrete [86]. Figure 3(c) shows plan and elevation views of the metal insert, which has diameter of 35 mm and a thickness of 18 mm. Sections of the disk have been removed to form surfaces (referred to as "cleats" by the inventor) which bear against the concrete when a torque is applied by a bolt inserted into the disk.

The torque test is analogous to the vane shear test used to measure the in-situ shear strength of soils. To perform this test, the torque bolt and disk are attached to the formwork and the concrete is placed. The torque bolt is treated with a release agent so that it can turn freely during the test and can be extracted for re-use after the test. The disk is provided with a grate, made with wires attached to a washer (see Fig. 3(c)), to prevent coarse aggregate particles from lodging within the cutouts in the disk. At the time of strength determination, a torque wrench is attached to the bolt, and the torque required to fracture that portion of the mortar which prevents the disk from rotating is measured. The perimeter of the disk, except for the bearing surface, is covered with a compressible tape to allow for the volume expansion during mortar fracture. The inventor claims that failure occurs by a combination of compression and shear on an inclined surface as shown in Fig. 3(c).

The correlation relationship between the ultimate torque and compressive strength has been obtained by placing bolt/disk assemblies in the ends of a 150 by 300 mm cylinders, performing the torque tests, and then measuring the compressive strength of the cylinders. The disks were embedded to a depth of 60 mm (2.4 in.) Stoll [86] showed that the torque test did not reduce the compressive strength of the cylinders because the disk was located within the constrained conical zone at the end of the cylinder. The mean coefficient of variation has been reported to be about 7%, which is comparable to other in-place tests.

Principle advantages of the Stoll Tork Test are that it uses inexpensive equipment and the test depth is not limited to the near surface layer. However, additional data are needed to understand the reliability of estimated compressive strength.

Maturity Method

The maturity method was proposed over 40 years ago as a means to estimate strength development under non-standard curing temperatures [60, 67, 82]. It was proposed that the temperature history could be used to compute an index that is indicative of the combined effects of time and temperature on strength gain. The function used to compute this index has become known as the "Nurse-Saul function," as follows:

$$M = \int_0^t (T - T_0)\, dt \tag{1}$$

where M = maturity index, °C-hours (or °C-days),
T = concrete temperature, °C, and
T_0 = datum temperature (usually taken to be 10 °C).

The index computed by Eq. (1) is often referred to as "maturity". However, the current terminology for this index is the "temperature-time factor" (ASTM C 1074 [4]).

Beginning in the mid 1970s, there has been renewed interest in the maturity method, and significant advancements have been made, including the development of an ASTM standard for using this technique. Chapter 5 in Malhotra and Carino [56] and the paper by Naik in these proceedings summarize the historical development of the method and provide examples of its application. Some of the author's ideas about the maturity method and recent research findings are summarized in this paper.

In 1977, Freiesleben Hansen and Pedersen [30] presented a new function for computing a maturity index. This function was based on the Arrhenius equation [10] and was used to compute the "equivalent age" of concrete as follows:

$$t_e = \int_0^t e^{\frac{-E}{R}\left(\frac{1}{T} - \frac{1}{T_r}\right)}\, dt \tag{2}$$

where t_e = the equivalent age at the reference temperature,
E = "activation energy," J/mole
R = universal gas constant, 8.314 J/K-mole,
T = absolute temperature of the concrete, degrees Kelvin,
T_r = absolute reference temperature, degrees Kelvin.

By using Eq. (2), the actual age of the concrete is converted to its equivalent age, in terms of strength gain, at the reference temperature. The reference temperature is usually taken to be the temperature for standard curing, which is 23 °C (73 °F) in North American practice. Byfors [13] found that this new maturity function was superior to the Nurse-Saul function in accounting for the combined effects of time and temperature.

Effect of temperature on strength gain -- The key parameter in Eq. (2) is the "activation energy" which describes the effect of temperature on the rate of strength development. In the early 1980s, the author began a series of studies to gain a better understanding of the maturity method [17]. From this work, a procedure was developed to obtain the "activation energy" of a given cementitious mixture. The procedure is based on determining the effect of curing temperature on the rate constant for strength development. The rate constant is related to the curing time needed to reach a certain fraction of the long-term strength, and it is obtained by fitting an appropriate equation to the strength versus age data acquired under constant temperature (isothermal) curing. The procedure to determine the "activation energy" includes the following steps:

- Cure mortar specimens at different constant temperatures.
- Determine compressive strengths at regularly spaced ages.
- Determine the value of the rate constant at each temperature by fitting a strength-age relationship to the each set of strength-age data.
- Determine the best-fit Arrhenius equation (to be explained) to represent the variation of the rate constant with the temperature.

By using the above procedure, the "activation energy" was determined for concrete and mortar specimens made with different cementitious materials [20, 88]. The results are summarized in Table 2. It was found that for concrete with water-cement ratio (W/C) = 0.45, the "activation energy" ranged from 30 and 64 kJ/mole; while for W/C = 0.60 it ranged from 31 to 56 kJ/mole, depending on the type of cement and additives.

The significance of the "activation energy" is explained further. In Eq. (2), the exponential term within the integral converts increments of actual curing time at the concrete temperature to equivalent increments at the reference temperature. Thus the exponential term can be considered as an *age conversion factor*, γ:

$$\gamma = e^{\frac{-E}{R}\left(\frac{1}{T}-\frac{1}{T_r}\right)} \tag{3}$$

Figure 4 shows how the age conversion factor varies with curing temperature for different values of the "activation energy". The reference temperature is taken as 23 °C. It is seen that for an "activation energy" of 30 kJ/mole, the age conversion factor is nearly a linear function of temperature. In this case, the Nurse-Saul equation would be a reasonably accurate maturity function to account for the combined effects of time and temperature, because the Nurse-Saul function assumes that the rate constant varies linearly with temperature [17]. For an "activation energy" of 60 kJ/mole, the age conversion factor is a highly non-linear function of the curing temperature. In this instance, the Nurse-Saul function would be an inaccurate maturity function. In summary, Fig. 4 shows the nature of the error in age conversion factor if the incorrect value of activation energy were used for a particular concrete mixture. The magnitude of the error would increase as the curing temperature was further from 23 °C.

The reader will have noticed that the term "activation energy" has been used within quotation marks. This is because the E-value that is determined when the rate constant is plotted as a function of the curing temperature is not truly an activation energy as implied by the Arrhenius equation. The following discussion is provided for those unfamiliar with the concept of activation energy or the origin of the Arrhenius equation.

The idea of "activation energy" was proposed by Svante Arrhenius in 1888 to explain why chemical reactions do not occur instantaneously when reactants are brought together, even though the reaction products are at a lower energy state [10]. Arrhenius proposed that before the lower energy state is achieved, the reactants must have sufficient energy to overcome an energy barrier separating the unreacted and reacted states. A physical analogy is brick standing upright. The brick will not instantaneously tip over to its lower energy state of lying horizontal. It must be pushed from the higher to the lower energy state. The energy required to push the brick from it upright position to the point of instability, after which the brick falls on its own, is the activation energy for this process.

For molecular systems, the reactant molecules are in constant motion and energy is transferred between them as they collide [10]. A certain number of molecules will acquire sufficient energy to surmount the energy barrier and form the lower energy reaction product. As the system is heated, the kinetic energy of the molecules increases and more molecules will surmount the barrier. Thus the rate of reaction increases with increasing temperature. Arrhenius observed that the rate constant, k, of many reactions increased with temperature according to what has since been called the Arrhenius equation, as follows:

$$k = A\, e^{-\frac{E}{RT}} \tag{4}$$

The term A is called the "frequency factor" and is related to the frequency of collisions and the probability that the molecules will be favorably oriented for reaction [10]. It can be seen that the age conversion factor given by Eq. (3) is simply the ratio of the rate constants at two different temperatures.

The Arrhenius equation was derived empirically from observations of homogeneous chemical systems undergoing a single reaction. Roy and Idorn [75] have noted that *"...workers... have cautioned that since cement is a multiphase material and also the process of cement hydration is not a simple reaction, homogeneous reaction kinetics cannot be applied."* Thus the "activation energy" obtained from strength gain data or degree of hydration data is not a true activation energy as originally proposed by Arrhenius. This is why quotation marks have been used.

The author believes that the Arrhenius equation happens to be one of several equations that can be used to describe the variation of the rate constant for strength gain (or degree of hydration) with curing temperature. This has been the motivation for proposing a simpler function than Eq. (2) to compute equivalent age [16, 20, 88]. It is suggested that the temperature dependence of the rate constant for strength gain can be represented by the following:

$$k = A_0\, e^{BT} \tag{5}$$

where A_0 = the value of the rate constant at 0 °C,
B = temperature sensitivity factor, 1/°C, and
T = concrete temperature, °C.

Using Eq. (5), the equation for equivalent age at the reference temperature T_r is as follows:

$$t_e = \int_0^t e^{B(T - T_r)}\, dt \tag{6}$$

A numerical study was performed to determine whether Eq. (5) can represent the temperature dependence of the rate constant as well as Eq. (3). Values of the rate constant were calculated at different temperatures using the Arrhenius equation (for assumed values of A and E), and Eq. (5) was fitted to those points using the method of least squares. Figure 5 shows an example of the points and the best-fit curve for an "activation energy" of 45 kJ/mole. It is seen that Eq. (5) is a good fit to the points. By performing these calculations for different values of "activation energy", it was found that there was a linear relationship between E and B as shown in Fig. 5(b). It is concluded that Eqs. (2) and (6) can be used interchangeably, with similar results, to calculate the equivalent age. However, the author believes Eq. (6) has the following advantages over Eq. (2):

- The temperature sensitivity factor, B, has more physical significance compared with the "activation energy": for each temperature increment of 1/B, the rate constant increases by a factor of approximately 2.7.
- Temperatures do not have to be converted to the absolute scale.
- Eq. (6) is a simpler equation.

The key to developing an accurate maturity function for a particular concrete mixture is to determine the variation of the rate constant with curing temperature. Strictly speaking, a rate constant represents the rate at which a chemical reaction occurs at a given temperature. However, in the context of this discussion, the rate constant is related to the rate of strength gain at a constant temperature, and it can be obtained from the equation of strength gain versus age. Thus it is necessary to consider some of the relationships that have been used to represent strength development of concrete.

Strength development relationships -- The author has used successfully the following hyperbolic equation for strength gain up to equivalent ages at 23 °C of about 28 days:

$$S = S_u \frac{k\,(t - t_0)}{1 + k\,(t - t_0)} \tag{7}$$

where S = strength at age t,
S_u = "limiting" strength,
k = rate constant, 1/day,
t_0 = age at start of strength development.

The basis of this equation has been explained elsewhere [17, 44]. This model assumes that strength development begins instantaneously at age t_0. Thus the gradual strength development during the setting period is not considered. The parameters S_u, k, and t_0 are obtained by least-squares curve fitting to strength versus age data. The limiting strength, S_u, is the asymptotic value of the strength for the hyperbolic function that fits the data. As will be discussed below, the best-fit value for S_u does not necessarily represent the actual long-term strength of the concrete, and that is why the quotation marks are used in the above definition. For the hyperbolic model, the rate constant has the following property: when the age beyond t_0 is equal to 1/k, the strength equals 50% of the "limiting strength," S_u.

A similar equation to Eq. (7) was also used by Knudsen [44] and Geiker [32] to represent the degree of hydration and development of chemical shrinkage as a function of age. However, Geiker [32] noted that Eq. (7) gave a poor fit for certain cementitious systems. It was found that the following version of the hyperbolic equation gave a better fit to that data than Eq. (7) [45]:

$$S = S_u \frac{\sqrt{k\,(t - t_0)}}{1 + \sqrt{k\,(t - t_0)}} \tag{8}$$

Knudsen [45] explained the differences between Eq. (7) and Eq. (8) in terms of the hydration kinetics of individual cement particles. Equation (7) is based on "linear kinetics," which means that the degree of hydration of an individual cement particle is a linear function of the product of time and the rate constant. Equation (8) is based on "parabolic kinetics" which means that the degree of hydration is a function of the *square root* of the product of time and the rate constant. Thus Eqs. (7) and (8) will be called the *linear hyperbolic* and *parabolic hyperbolic* models.

Freiesleben Hansen and Pedersen [31] proposed the following exponential equation to represent strength development of concrete:

$$S = S_u \, e^{-\left(\frac{\tau}{t}\right)^{\alpha}} \tag{9}$$

where τ = a time constant and
α = a shape parameter.

This equation can model the gradual strength development occurring during the setting period and it is also asymptotic to a limiting strength. The time constant τ represents the age at which the strength has reached 0.37 S_u. Thus the value of $1/\tau$ is the rate constant for this equation. The shape parameter α affects the slope of the curve during the acceleratory period (following the induction period) and it affects the rate with which the strength approaches the limiting strength.

Examples are presented to illustrate the performance of these models in representing actual strength development data. Figure 6(a) shows strength data for mortar cubes cured at room temperature and tested at ages from 0.4 to 56 days. Figure 6(b) shows data for standard-cured concrete cylinders tested at ages from 7 days to 3.5 years [15]. The curves are the best-fit curves for Eqs. (7), (8), and (9). For the mortar data, the linear hyperbolic function and the exponential function fit the data well, and the curves in Fig. 6(a) are nearly indistinguishable. For the concrete data, the parabolic hyperbolic function and the exponential function fit the data well and these curves cannot be distinguished in Fig. 6(b).

The results shown in Fig. 6 highlight the capabilities of the various strength-age functions. The linear hyperbolic function appears to be a good model for strength development up to about 28 days (equivalent age) but not for later ages. The parabolic hyperbolic model appears to be better suited for modeling later age strength gain. The exponential model appears to be capable of modeling strength gain over the full spectrum of ages.

The inherent limitation of the linear hyperbolic function can be understood by considering the ratio of the "limiting strength" to the 28-day strength. If the t_0-term in Eq. (7) is neglected, the following equation is obtained for this ratio:

$$\frac{S_u}{S_{28}} = \beta = 1 + \frac{1}{28\,k} \tag{10}$$

Thus the value of β is directly related to the rate constant. A higher value k results in a lower value of β, which means a smaller difference between the "limiting strength" and the 28-day strength. The rate constant is, in turn, controlled primarily by the initial rate of strength development. The fact is that the ratio of actual long-term strength of concrete to the 28-day strength does not obey Eq. (10). This means that value of S_u obtained by fitting the linear hyperbolic model to strength-age data will be lower than the actual long-term strength of concrete if allowed to cure for a long time.

Based on the above discussion, one might conclude that the exponential model given by Eq. (9) is the best model to use for determining the rate constant at a particular curing temperature. This would be correct if the shape parameter, α, were independent of the curing temperature. Recent unpublished results of tests at NIST show that this is not the case. Thus a maturity function based solely upon the variation of the rate constant $(1/\tau)$ with temperature would not be able to account accurately for the combined effects of time and temperature on strength development.

Which strength-gain model should be used? The maturity method is used typically to monitor strength development during construction. Therefore, it is not necessary to model accurately the strength gain at later ages. Thus the author believes that the linear hyperbolic model can be used to analyze strength data, up to a 28-day equivalent age, to determine the variation of the rate constant with curing temperature. Knudsen[3] suggests that the linear hyperbolic model is suitable up to a degree of hydration of 85%.

Estimating strength -- The final discussion deals with estimating strength by the maturity method. The maturity method is generally used to estimate the in-place strength of concrete by using the in-place maturity index and a previously established relationship between maturity index and strength. This assumes that a given concrete possesses a unique relationship between strength and the maturity index. This assumption would be acceptable if the long-term strength of concrete were independent of the curing temperature, but this is not the case. It is known that the long-term strength is affected by initial temperature of the concrete. Thus if the same concrete mixture were used for a cold weather placement and a hot weather placement, the strength would not be the same for a given maturity index. It is proposed that the correct application of the maturity method is to estimate *relative strength*. Tank and Carino [88] proposed the following "rate constant" model of relative strength development in terms of equivalent age:

$$\frac{S}{S_u} = \frac{k_r\,(t_e - t_{0r})}{1 + k_r\,(t_e - t_{0r})} \tag{11}$$

where k_r = value of the rate constant at the reference temperature, and
t_{0r} = age at start of strength development at the reference temperature.

However, the previous discussion has shown that, for the linear hyperbolic model, the ratio S/S_u may not indicate the true fraction of the long-term strength because the calculated value of S_u may not be the actual long-term strength. This deficiency can be overcome by expressing relative strength as a fraction of the strength at an equivalent age of 28-days. By using the definition of β in Eq. (10), the relative strength gain equation would be as follows:

$$\frac{S}{S_{28}} = \beta\,\frac{k_r\,(t_e - t_{0r})}{1 + k_r\,(t_e - t_{0r})} \tag{12}$$

The value of β would be obtained by fitting Eq. (7) to data of strength versus equivalent age. Then the value of S_u is divided by the estimated strength, from the best-fit curve, at an equivalent age of 28 days.

To summarize, the following points represent the author's ideas about the maturity method:

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[3] T. Knudsen, The Technical University of Denmark, April 1985, personal communication

- There is no single maturity function that is applicable to all concrete mixtures. The applicable maturity function for a given concrete can be obtained by determining the variation of the rate constant with the curing temperature.
- The linear hyperbolic function can be used to analyze strength age data to obtain the rate constants at different curing temperatures.
- Equation (5) can be used to represent the variation of the rate constant with curing temperature. The temperature sensitivity factor governs the rate at which the rate constant increases with temperature and is analogous to the "activation energy" in the Arrhenius equation.
- The equivalent age can be calculated from the temperature history using Eq. (6).
- The maturity method is more reliable in estimating relative strength development rather than absolute strength.

METHODS FOR FLAW DETECTION

The other types of NDT methods are those for flaw detection, where the term "flaw" can include voids, honeycombing, delaminations, cracks, lack of sub-base support, etc. Recent research and development efforts on these methods have far exceeded those for methods to estimate strength. The research impetus has come primarily from the transportation industry, since much of the highway infrastructure is in need of repair as a result of natural aging or damage resulting from corrosion of reinforcing steel or deterioration of concrete.

The techniques for flaw detection are based generally on the following simple principle: the presence of an internal anomaly interferes with the propagation of certain type of waves. By monitoring the response of the test object when it is subjected to these waves, the presence of the anomaly can be inferred. The interpretation of the results of these types of NDT methods usually requires an individual who is knowledgeable both in concrete technology and in the physics governing the wave propagation. This section of the paper reviews the following techniques: (1) infrared thermography, (2) ground penetrating radar, and (3) stress-wave propagation methods. Radioactive and nuclear methods are not discussed, but a state-of-art summary is provided by Mitchell in Chapter 10 of Malhotra and Carino [56].

Infrared Thermography

Infrared thermography is based on the principle that the presence of an anomaly having a lower thermal conductivity than the surrounding material will interfere with the flow of heat and alter the surface temperature distribution. By measuring the surface temperature, the presence of the anomaly can be detected. Representative values of thermal conductivity coefficients are as follows [36]:

Material	Thermal Conductivity (J/s•m•°C)
Steel	46
Ice	1.7
Concrete	0.8
Air	0.024

In civil engineering applications, the method is used primarily for detecting delaminations in reinforced concrete pavements and bridge decks resulting from corrosion of the steel. The majority of the early research work for this application was performed by the Ontario Ministry of Transportation and Communication [57]. In 1988, ASTM Standard C 4788 [4] was published which covers the use of the method for determining the locations of delaminations in exposed and overlaid concrete bridge decks. A review of the method and case histories are provided by Weil in Chapter 13 of Malhotra and Carino [56].

The principle of infrared thermography is illustrated in Fig. 7. The presence of a void in the concrete slab has a local insulating effect which disrupts the heat flow through the slab and affects the surface temperature. When heat flows into the slab, the area above the void is warmer than the surrounding area, and when heat flows out of the slab the area above the void is cooler. By measuring the surface temperature distribution, one can infer the presence of the void. Hence, to apply infrared thermography, there must be a heat flow condition through the test object, and a means is required to measure small differences in surface temperature.

The required heat flow condition can be created artificially by using heating lamps, or it can occur naturally by solar heating (heat flow into structure) and night-time cooling (heat flow out of structure). The latter method is obviously the economical approach. The suggested best time for doing infrared surveys is two to three hours after sunrise or after sunset [56].

Even with the proper heat flow conditions, not all delaminations are detectable. Maser and Roddis [59] performed analytical studies to gain an understanding of the factors affecting the differences in the surface temperature of a solid concrete slab and a slab with a delamination. It was found that the maximum differential temperature decreased as the depth of the delamination increased and as the width decreased. Also, a water-filled delamination resulted in nearly identical surface temperatures as in a solid slab.

In infrared thermography, differences in surface temperature are measured by using an imaging infrared scanner, a device similar to a video camera, which measures the amount of infrared radiation emitted by a surface. The underlying principle of this measurement method is as follows: Objects at temperatures above absolute zero emit electromagnetic radiation, whose wavelength depends on the temperature. As the temperature increases, wavelength become shorter, and, at sufficiently high temperature, the radiation is in the visible spectrum. This is the operating principle of incandescent light bulbs. At room temperature range, the wavelength of the radiation emitted by surfaces is on the order of 10 μm, which is in the infrared region of the spectrum. This radiation cannot be detected by the naked eye. The infrared scanner has a detector which "sees" the infrared radiation. The detector output is related to the amount of incident radiant energy.

The energy emitted by a surface is related to its temperature according to the Stefan-Boltzman law [36]:

$$R = e\,\sigma\,T^4 \tag{13}$$

where R = rate of energy radiation per unit area of surface, $\mathrm{W/m^2}$,
e = the emissivity of the surface,
σ = the Stefan-Boltzman constant, $5.67 \times 10^{-8}\ \mathrm{W/m^2 \cdot K^4}$
T = absolute temperature of the surface, K.

By proper calibration, the output from the infrared sensor can be converted to a temperature.

The emissivity is a characteristic of the material and surface texture, and has a value ranging from 0 to 1. Since the rate of energy radiation depends on temperature and emissivity, care must be exercised in interpreting thermographic images to assure that apparent temperature differences are not caused by differences in emissivity. The emissivity of bridge deck surfaces can be affected by the type of texturing, oil spots, tire marks, paint, and loose debris.

Equipment for performing a thermographic survey includes: an infrared scanner with associated hardware capable of producing a television image of the temperature distribution of the scanned surface, a conventional video camera to provide a visual record for comparison with the infrared record; video recorders to record infrared and conventional video images; analog to digital converters to transform the video images into digital data; a computer system and software for data storage and signal enhancement; and a distance referencing system to correlate the infrared scan with position on the bridge deck [48, 56]. The equipment is typically contained in a mobile van which travels along the roadway while data are recorded. The resolution of the infrared scanner is improved by lowering its temperature, therefore, a liquid nitrogen cooling system is used to cool the sensor. Available equipment allows resolution of differences in surface temperature as low as 0.1 °C.

The following is a summary of the procedure in ASTM D 4788 [4] for performing an infrared thermographic survey to detect delaminations in bridge decks:

- Remove debris from surface.
- Allow the surface to dry for at least 24 hours before testing.
- There must be at least 0.5 °C difference between the surface temperatures in areas above delaminations and in sound concrete. A minimum of 3 hours of direct sunlight are generally sufficient to establish this temperature difference. A contact thermometer with a minimum resolution of 0.1 °C is used to determine whether the minimum temperature difference has been established.
- Do not test when the wind speed exceeds 50 km/h.
- Do not test when the ambient air temperature is less than 0 °C, because ice in delaminations will give false indications of sound concrete. As a guide, an ambient temperature rise of 10 °C, 4 hours of sunshine, and a wind speed below 25 km/h should result in accurate data on bare concrete surfaces during winter. For asphalt-covered concrete, at least 6 hours of sunshine are necessary during winter.
- Collect data with the van moving at speeds not greater than 15 km/h.

The results of the inspection are usually reported in terms of delaminated area and percentage of delaminated area. After the delaminated areas are identified in the infrared images, the visible video images should be compared to assure that apparent temperature differences were not due to emissivity changes [48]. The ASTM standard states that 80 to 90% of the delaminations in a bare bridge deck can be located by this method. It has also been found that the inspection of the same deck by four different operators resulted in a variation of ±5% of the known delaminated area.

In summary, instrumentation and computer software have been developed so that inspection of bridge decks is a fairly routine procedure [48]. Trained individuals are required to assure that meaningful data are collected and that the data are correctly interpreted. Infrared thermography is a "global" inspection method. This permits large surface areas to be scanned in a short period of time, which is an advantage over other methods to be discussed.

Radar

Radar (acronym for RAdio Detection And Ranging) is analogous to the sonar technique for determining the distance to an objective, except that electromagnetic waves (radio waves or microwaves) are used instead of sound. While the early uses of the technique were for military applications, radar techniques are now used in a variety of fields, such as weather, aerial mapping, and civil engineering applications. The earliest civil engineering applications were probing into soil to detect buried pipelines and tanks. This was followed by studies to detect cavities below airfield pavements, and more recently, the inspection of bridge decks [14, 26].

In civil engineering applications, relatively short distances are involved compared with other uses of radar. As a result, devices for these applications emit very short pulses of electromagnetic waves (microwaves). For this reason the technique is often called "short-pulse radar" or "impulse radar." Others call it "ground penetrating radar" (GPR). In this paper it will be called GPR. This section discusses the principles of GPR, the instrumentation that is used, and some of the inherent difficulties in using the method. A recent review of GPR by Clemeña may be found in Chapter 11 of Malhotra and Carino [56].

The operating principle of GPR is illustrated in Fig. 8(a). An antenna on the surface of the test object sends out a short duration pulse (on the order of nanoseconds) of electromagnetic waves. The pulse travels through the test object and when it encounters an interface between dissimilar materials, some of the energy is reflected back toward the antenna as an "echo". The antenna receives the echo and generates an output signal. By measuring the time from the start of the pulse until the reception of the echo, one can determine the depth of the interface.

The amplitude of reflection at an interface depends on the difference between the relative dielectric constants of the two materials and is given by the following equation [56]:

$$\rho_{1,2} = \frac{\sqrt{\epsilon_{r1}} - \sqrt{\epsilon_{r2}}}{\sqrt{\epsilon_{r1}} + \sqrt{\epsilon_{r2}}} \tag{14}$$

where $\rho_{1,2}$ = reflection coefficient,
ϵ_{r1} = relative dielectric constant of material 1, and
ϵ_{r2} = relative dielectric constant of material 2.

By definition, the relative dielectric constant of air equals 1, and typical values for other materials are given in Fig. 8(a). The relative dielectric constants for materials such as concrete and soil depend on the moisture content and ionic concentrations [56]. As the moisture content increases, the values of ϵ_r increase. Figure 8(b) shows how the reflection coefficient at an interface varies with the ratio of the relative dielectric constants of the two materials. It is seen that when the value of ϵ_r of material 2 is greater than that of material 1, the reflection coefficient is negative. This signifies that there is a phase reversal in the reflected wave, which means that the positive part of the wave is reflected as a negative part. At a metal interface, such as between concrete and steel reinforcement, there is complete reflection, and the reflected wave has reversed polarity.

The depth of the reflecting interface is obtained from the measured round trip travel time and the speed of the electromagnetic wave, C, which is dependent upon the relative dielectric constant:

$$C = \frac{C_0}{\sqrt{\epsilon_r}} \tag{15}$$

where C_0 = speed of light in air (3×10^8 m/s) and
ϵ_r = relative dielectric constant.

If the round trip travel time is Δt, the depth, D, would be

$$D = \frac{C\,\Delta t}{2} \tag{16}$$

Equations (14) through (16) form the basis for using GPR to inspect concrete structures.

Typical instrumentation for GPR includes the following main components: an antenna unit, a control unit, a display device, and a storage device. The antenna emits the electromagnetic pulse and receives the echoes. The length of the pulse is largely controlled by the antenna design. Longer pulses are associated with longer wavelengths (or lower frequency) and have more penetrating ability, but poorer resolution, than shorter pulses. Typically, an antenna with a center frequency of about 1000 MHz is used to inspect pavements and bridge decks, and the pulse length is about 1 nanosecond. In air such a pulse is about 0.3 m long, and in concrete the length would depend on the value of ϵ_r. For $\epsilon_r = 6$, the pulse length would be about 120 mm. To be able to measure depths accurately, the echo must arrive after the initial pulse has ceased. Therefore, the round trip travel path must exceed the pulse length. For $\epsilon_r = 6$, the minimum depth that can be measured accurately is about 60 mm. As ϵ_r increases, the minimum measurable depth decreases. The pulse is attenuated as it travels through the test object, and there is a limit to the thickness that can be inspected. For concrete, the depth of penetration would depend on the characteristics of the GPR system, the moisture content, and the amount of reinforcement. With increasing moisture content and amount of reinforcement, the penetration decreases. For dry unreinforced concrete, the maximum penetration of the pulse produced by a 1000 MHz antenna is about 0.6 m [56]. In Fig. 8(a), the antenna is in contact with the test objects. It is also possible to use a non-contact "horn" antenna. In this case, the received signal includes an echo from the concrete surface.

The control unit is the heart of a GPR system. It controls the repetition frequency of the pulse, provides the power to emit the pulse, acquires and amplifies the received signal, and provides output to a display device. Data are usually stored in an analog recorder and played back for later analysis and interpretation.

Display devices include oscillographs, which plot the recorded waveforms as a "waterfall plot," or graphic facsimile recorders. Computer software is also available to permit sophisticated signal processing of the data to aid in interpretation. The operation of the graphic recorder is discussed further because it is commonly used in the field. Figure 9(a) shows an antenna emitting a radar pulse into a test object containing a void. The shape of the emitted pulse is sketched above the antenna. Figure 9(b) shows the waveform from the receiving side of the antenna. The vertical axis represents time, or it can be transformed to depth by knowing ϵ_r and using Eq. (16). The received signal at the start of the waveform represents the transmitted pulse, which is picked up directly by the receiving side of the antenna. The second received signal is the echo from the void, and the third is the echo from the bottom boundary of the test object.

The output of the graphic recorder is obtained by a technique known as "peak plotting," which is illustrated in Fig. 9(b). First, the operator selects a threshold voltage range. When the amplitude of the received signal goes beyond the threshold range, the pen of the graphic recorder plots a solid line on recording paper. The line is plotted in varying shades of gray depending on the actual amplitude of the signal. Thus the antenna output is represented on the graphic recorder as a series of dashes as shown in Fig. 9(b). Note each echo is associated with two dashes. The actual number of dashes depends on the number of cycles in the emitted pulse and the threshold level. This is an important point to understand for proper interpretation of GPR results. As the antenna is moved along the surface, the output is displayed on the graphic recorder. The paper on the recorder moves at a constant speed that is independent of the speed of the antenna motion. The resulting "picture" on the graphic recorder represents a cross-sectional view of the test object, as illustrated in Fig. 9(c). The test equipment provides a means for correlating the position of the antenna during the scan with the location on the paper record. Thus it is possible to determine the depth and approximate size of the reflecting interface.

As was mentioned, metals are strong reflectors of electromagnetic waves. This makes GPR very effective in locating buried metal objects, such as reinforcing bars and conduit. Reinforcing bars result in characteristic patterns in the graphic recorder output, which make them relatively easy to locate. There are two main factors leading to these patterns, which are discussed with the aid of Fig. 10. First of all, an antenna behaves like a flashlight in that the beam of radiation has a conical shape. Thus a reinforcing bar produces echoes before the antenna passes directly over the bar; however, the apparent depth of the bar will be larger than the actual cover because of the inclined travel path. Secondly, the pulse undergoes multiple reflections, or reverberations, between the reinforcing bar and the surface, and the output will show multiple echoes from the same bar. The resulting characteristic pattern is shown on the right side of Fig. 10. The top of the bar is associated with the uppermost part of the arched bands.

When multiple reinforcing bars are present, there will be multiple arch patterns. As the bars become closer together, the arch patterns overlap. Below a certain spacing, the individual bars can no longer be discerned, and the echo pattern is similar to the case of a solid embedded steel plate. Arcone [3] reports that when the spacing of a gridwork of metal bars is smaller than one-half of the predominant wavelength in the pulse, almost total reflection of the pulse occurs.

Simple methods do not exist to determine bar size from graphic recorder output. Research is underway to better understand the interactions of GPR with cracks, voids and reinforcing bars [58]. The objective of these studies is to develop procedures to use the recorded data to construct an image of the interior of the concrete. Some advances in this direction have occurred in France, where a prototype system has been developed that uses tomographic techniques to reconstruct the reinforcement layout in concrete [73]. The system is called the "Microwave Camera," and it uses an array of receivers to record the reinforcing bar echoes from different directions. The received signals are processed with a mini-computer to create an image of the bars in the concrete. For improved resolution, the emitted pulses are in the range of 7 to 13 GHz, which limits the depth of penetration to about 80 mm [73]. The Microwave Camera is able to detect layered bars, but the reported resolution in the location of the bars and their size is not very good and further development is needed.

The detection of delaminations in reinforced bridge decks using GPR is not straightforward. In studies by Maser [59], it was found that a 3-mm air gap in concrete produced little noticeable effect in the received waveform. However, the addition of moisture to the simulated crack resulted in stronger reflections which could be noticed in the waveform. It was also found that the presence of chlorides in moist concrete resulted in high attenuation, because of the increased relative dielectric constant. Thus the ability to detect delaminations will depend on the in-situ conditions at the time of testing. In addition, the reflections from reinforcing bars are much stronger than from a delamination, and it is difficult to "see" the delamination.

Due to difficulties in using GPR in reinforced concrete, standardized test procedures for flaw detection do not exist. However, an ASTM standard has been developed (Method D 4748 [4]) to measure the thickness of the upper layer of a multi-layer pavement system. Basically, the technique involves measuring the transit time of the pulse though the pavement layer, and using relationships similar to Eqs. (15) and (16) to calculate the layer thickness. The procedure is based on using a non-contact horn antenna, and some modifications are required for measurements with a contact antenna. The accuracy of the calculation is dependent on the accuracy of the value of the relative dielectric constant. Thus it is necessary to take occasional cores to determine the appropriate value for the pavement materials. The user is cautioned against using the method on saturated concrete, because of the high attenuation and limited penetration of the pulse. It is stated that inter-operator testing of the same materials resulted in thickness measurements within ±5 mm of the actual thickness. Finally, it is noted that reliable interpretation of received signals can only be performed by an experienced data analyst.

While the majority of the applications of GPR have dealt with locating reinforcing bars in structures, locating delaminations in bridge decks, and measuring the thickness of pavement layers, there are other potential uses. Since the dielectric properties of a material like concrete are strongly dependent on the moisture content, microwave measurements can be used to monitor the progress of hydration [Chapter 11 in [56], 70]. This is made possible because the relative dielectric constant of free water is much higher than that of chemically bound water. Clemeña has also reported on potential applications of microwave measurements to determine water content of fresh concrete [56].

Stress Wave Propagation Methods[4]

Tapping an object with a hammer or steel rod (sounding) is one of the oldest forms of nondestructive testing based on stress wave propagation. Depending on whether the result is a high-pitched "ringing" sound or a low frequency "rattle" sound, the integrity of the member can be assessed. The method is subjective, as it depends on the experience of the operator, and it is limited to detecting near surface defects. Despite these inherent limitations, "sounding" is a useful method for detecting near-surface delaminations, and it has been standardized by ASTM as Practice D 4580 [4].

In NDT of metals, the ultrasonic pulse-echo (UP-E) method has proven to be a reliable method for locating small cracks and other defects (Fig. 11(a)). The principle of UP-E is similar that of GPR, except that an electro-mechanical transducer is used to generate a pulse of ultrasonic stress waves instead of electromagnetic waves. Stress wave reflections occur at boundaries separating materials with different densities and elastic properties (these determine the acoustic impedance of a material). At the interface between a solid and air, nearly all of the incident energy is reflected. Thus stress-wave propagation methods are particularly suited for detecting defects such as voids and cracks.

Attempts to use UP-E equipment designed for metal inspection to test concrete have been unsuccessful because of the heterogeneous nature of concrete. The presence of paste-aggregate interfaces, air voids, and reinforcing steel result in a multitude of echoes which obscure those from real defects. However, in the last 10 to 15 years there has been considerable progress in the development of usable techniques based on stress wave propagation. This section reviews the principles of the various methods. A more comprehensive review is provided in Chapter 12 of Malhotra and Carino [56].

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[4] The author acknowledges the assistance of Dr. Mary Sansalone, Cornell University, in preparing portions of this section.

Ultrasonic pitch-catch -- In the UP-E method, the same transducer sends pulses of ultrasonic waves and receives the echoes. For this approach to work, the pulse must be shorter than the round trip travel time of the first echo. Short duration pulses (of the order of 1 microseconds) produce very high frequency waves which have limited penetration. Thus it has been impossible to produce a true pulse-echo system for concrete. However, success has been achieved in the development of ultrasonic "pitch-catch" systems (see Fig. 11(b)). In this case, separate transducers on the same surface are used to send and receive the signal [9, 25, 40, 89] In general, transmitting transducers with a resonant frequency between 100 and 200 kHz have been used. The output of the receiving transducer is displayed on an oscilloscope, and the time from the initiation of the pulse until the arrival of the echo is determined. Figure 12 shows an example of the waveform captured by an oscilloscope using a recently developed pitch-catch system [2, 89]. In this case, the echo comes from the opposite face of a 230-mm thick slab. If the P-wave speed[5] is known, the thickness of the slab or depth of a defect can be determined by using an equation analogous to Eq. (16). To date, pitch-catch systems are capable of testing concrete structures up to about 300 mm thick. An experienced operator is needed to correctly identify the arrival time of the echo on the oscilloscope display.

Impact methods -- The greatest success in the application of stress wave methods has been to use mechanical impact to the generate the stress pulse. This eliminates the need of bulky transmitting transducers and the associated electronics. Impact results in a high energy pulse resulting in high penetration of the stress waves. Several techniques have been developed which are similar in principle, but differ in the specific instrumentation and signal processing methods that are used [28, 39, 69, 76, 81, 84, 85].

Figure 13 is a schematic of an impact test. The principle is analogous to other echo methods that have been discussed. Impact on the surface produces a disturbance which travels into the object along spherical wavefronts as P- and S-waves. In addition, a surface wave (R-wave) travels away from the impact point. The P- and S-waves are reflected by internal defects (difference in elastic constants and density) or external boundaries. When the reflected waves, or echoes, return to the surface, they produce displacements which are measured by a receiving transducer. If the transducer is placed close to the impact point, the response is dominated by P-wave echoes [76]. Using "time domain" analysis, the time from the start of the impact to the arrival of the P-wave echo is measured, and the depth of the reflecting interface can be determined if the P-wave speed is known.

The first successful applications of impact methods occurred in geotechnical engineering [28, 85] to evaluate the integrity of concrete piles and caissons. The technique became known as the "sonic-echo" or "seismic-echo" method. The long length of the foundation structures allowed sufficient time separation between the generation of the impact and the echo arrival, and determination of round-trip travel times was relatively simple [50, 68]. The impact response of thin concrete members, such as slabs and walls, is more complicated than that of long slender members. Recent work by Sansalone and Carino [76] have led to the development of a successful technique for flaw detection in relatively thin concrete structures. The technique is known as the "impact-echo method."

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[5] P-waves are associated with normal stress and S-waves are associated with shearing stress.

A key development leading to the success of the impact-echo method was the use of "frequency analysis" instead of time domain analysis of the recorded waveforms [18]. The principle of frequency analysis is as follows: The P-wave produced by the impact undergoes multiple reflections between the test surface and the reflecting interface. Each time the P-wave arrives at the test surface, it causes a characteristic displacement. Thus the waveform has a periodic pattern, that is dependent on the frequency, f, of the P-wave arrival which is given by

$$f = \frac{C_p}{2\,D} \tag{17}$$

where C_p is the P-wave speed and D is the depth of the reflecting interface. This frequency is termed the "thickness frequency."

To apply frequency analysis, the recorded waveform is transformed into the frequency domain by using the fast Fourier transform technique [8]. The computed amplitude spectrum shows the dominant frequencies in the waveform. For slab-like structures, the thickness frequency will usually be the dominant peak in the spectrum. The value of the peak frequency in the amplitude spectrum can be used to determine the depth of the reflecting interface by expressing Eq. (17) as follows:

$$D = \frac{C_p}{2\,f} \tag{18}$$

Figure 14 illustrates the use of frequency analysis of impact-echo tests. Figure 14(a) shows the amplitude spectrum from a test over a solid portion of a 0.5-m thick concrete slab. There is a frequency peak at 3.42 kHz, which corresponds to multiple P-wave reflections between the bottom and top surfaces of the slab. Using Eqs. (17) or (18) and solving for C_p, the P-wave speed is calculated to be 3420 m/s. Figure 14(b) shows the amplitude spectrum from a test over a portion of the slab containing a disk-shaped void [79]. The peak at 7.32 kHz results from multiple reflections between the top of the slab and the void. Using Eq. (18), the calculated depth of the void is 3420/(2 x 7320) = 0.23 m, which compares favorably with the known distance of 0.25 m.

In the initial work leading to the impact-echo method [76], it was noted that the duration of the impact was critical in determining the success of the method. As shown in Fig. (13), the force-time history of an impact may be approximated as a half-cycle sine curve, and the duration of the impact is the "contact time." The contact time determines the frequency content of the stress pulse generated by the impact [18]. As an approximation, the highest frequency component of significant amplitude equals the inverse of the contact time. In order to accurately locate shallow defects, the stress pulse must have frequency components greater than the frequency corresponding to the flaw depth (Eq. (18)). For example, for a P-wave speed of 4000 m/s, a pulse with a contact time shorter than 100 μs is needed to determine the depth of defects shallower than about 0.2 m. Various impact sources have been used for impact testing. In evaluation of piles, hammers can be used to produce energetic impacts with long contact times (on the order of 1 ms). Such impacts are acceptable for testing long, slender structures but not for slabs or walls. For testing slabs form 0.15 to 0.5 m thick, steel spheres and spring-loaded spherically-tipped impactors have been used successfully. Steel spheres are convenient impact sources because the contact time is proportional to the diameter of the sphere [35].

The impact-echo method been successful in detecting a variety of defects, such as voids and honeycombed concrete in members, delaminations in bare and overlaid slabs, and voids in tendon ducts [21, 76, 78, 79,80]. Experimental studies have been supplemented with analytical studies to gain a better understanding of the propagation of transient waves in bounded solids with and without flaws [77, 49, 50]. Recent work [51] has extended the application of the method to prismatic members, such as columns and beams. It has been found that reflections from the perimeter of these members cause complex modes of vibration. As a result, the amplitude spectra have many peaks, and the depth of the member is not related to the dominant frequency in the spectrum according to Eq. (18). Nevertheless, it has been shown that defects can still be detected within beams and columns.

As with most methods for flaw detection in concrete, experience is required to interpret stress-wave test results. A recent advance in the interpretation of impact-echo results from tests of slab-like structures has been the application of an artificial intelligence technique known as a "neural network" [81]. In this technique, a computer program is "trained" to recognize amplitude spectra associated with flawed and unflawed structures. After this "training", the program can be used to classify the results of tests on a structure under investigation. This technique is being incorporated into the development of the first commercial impact-echo test system[6].

Another variant of the impact method is known as "impulse-response" [28, 39, 69, 84]. In this approach, the force-time history of the impact as well as the response of the structure are measured. By a signal processing technique, the measured response and force history are used to compute the characteristic impulse response spectrum of the structure (see Chapter 12 of [56]). The impulse response spectrum of a structure depends on its geometry, the support conditions, and the existence of flaws or cracks.

Depending on the measured quantity of the structural response (displacement, velocity or acceleration), the impulse response spectrum has different meanings. Typically, velocity is measured and the resulting impulse-response spectrum has units of velocity/force which is referred to as "mobility," and the spectrum is called a "mobility plot." At frequency values corresponding to resonant frequencies of the structure, mobility values are maximum. In the testing of piles, the mobility plot has a series of peaks which correspond to the fundamental and higher longitudinal modes of vibration. The difference between any two adjacent peaks, Δf, is equal to the fundamental longitudinal frequency [39]. The length of the pile can be calculated by using Δf in place of f in Eq. (18).

To illustrate how the method works, impulse-response spectra obtained from two test piles having the same dimensions are shown in Fig. 15 [68]. Figure 15(a) is the response spectrum obtained from a sound pile. The P-wave speed in this pile was 4140 m/s. The fundamental longitudinal frequency of 138 kHz was calculated by determining the average frequency difference between four successive peaks. The pile length was calculated to be 15.0 m, while the known length was 15.2 m. For comparison, Fig. 15(b) shows the response spectrum from a pile which contained a full- width defect at a depth of 9.8 m. The P-wave speed in this pile was 4200 m/s. The fundamental frequency of 193 Hz was estimated by averaging the difference between three successive frequency peaks shown in Fig. 15(b). The depth to the reflecting interface was calculated to be about 11 m. In addition to providing information on the length of a pile, the impulse response function can also indicate the dynamic stiffness of the pile-soil structure. The initial slopes of the spectra in Fig. 15 are inversely related to the dynamic stiffnesses. Thus the presence of the void is indicated by a reduced pile length and an increased dynamic stiffness.

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[6] Sansalone, M. and Pratt, D.G, "Non-destructive Materials Testing Apparatus and Technique for use in the Field," U.S. Patent Application

Spectral Analysis of Surface Waves -- The technique known as "spectral analysis of surface waves" is an impact method which uses the information contained in the surface wave to determine the stiffness profiles in layered systems, such as pavements [37]. Figure 16 shows the configuration for SASW testing [64]. Two receivers are used to monitor the movement of the surface due to the R-wave produced by the impact. The received signals are processed and a complex calculation scheme is used to infer the stiffnesses of the underlying layers.

Just as the impact is composed of a range of frequency components, the R-wave also contains a range of components of different frequencies or wavelengths. (Note, the product of frequency and wavelength equals wave speed.) This range depends on the contact time of the impact; a shorter contact time results in a broader range. The longer wavelength (lower frequency) components penetrate more deeply, and this is the key to using the R-wave to gain information about the properties of the underlying layers [74]. In a layered system, the propagation speed of these different components is affected by the wave speed in those layers through which the components propagate. A layered system is a dispersive medium for R-waves, which means that different frequency components in the R-wave propagate with different speeds, which are called "phase velocities."

Phase velocities are calculated by determining the time it takes for each frequency (or wavelength) component to travel between the two receivers. These travel times are determined from the phase difference of the frequency components arriving at the receivers [56, 65]. The phase differences are obtained by computing the "cross power spectrum" of the signals recorded by the two receivers. The phase portion of the cross-power spectrum gives phase differences (in degrees) as a function of frequency. The phase velocities are determined as follows:

$$C_{R(f)} = X \frac{360}{\phi_f} f \tag{19}$$

where $C_{R(f)}$ = surface wave speed of component with frequency f,
X = distance between receivers (see Fig. 16), and
ϕ_f = phase angle of component with frequency f.

The wavelength, λ_f, corresponding to a component frequency is calculated using the following equation:

$$\lambda_f = X \frac{360}{\phi_f} \tag{20}$$

By repeating the calculations in Eqs. (19) and (20) for each component frequency, a plot of phase velocity versus wavelength is obtained. Such a plot is called a "dispersion curve." Figure 17 shows an example of a dispersion curve obtained by Nazarian and Stokoe [64] from tests on a concrete pavement. The short wavelength components have higher speeds because they correspond to components travelling only through the concrete slab.

A process called "inversion" is used to obtain the approximate stiffness profile at the test site from the experimental dispersion curve [65]. The test site is modelled as layers of varying thickness. Each layer is assigned a density and elastic constants. Using this information, the solution for surface wave propagation in a layered system is obtained and a theoretical dispersion curve is calculated for the assumed layered system. The theoretical curve is compared with the experimental dispersion curve. If the curves match, the problem is solved and the assumed stiffness profile is correct. If there are significant discrepancies, the assumed layered system is changed or refined and a new theoretical curve is calculated.

This process continues until there is good agreement between the theoretical and experimental curves.

The general configuration for SASW testing was shown in Fig. 16. For reliable results, tests are repeated with different receiver spacings [64]. The receivers are first located close together, and the spacing is increased by a factor of two for subsequent tests. As a check on the measured phase information for each receiver spacing, a second series of tests is carried out by reversing the position of the source. Typically, five receiver spacings are used at each test site. For tests of concrete pavements, the closest spacing is usually about 0.15 m [64].

The SASW method has been used to determine the stiffness profiles of soil sites and of flexible and rigid pavement systems. Case studies are reported by Nazarian and Stokoe [65], and one example is summarized in Malhotra and Carino [56]. Recently the method has been extended to the measurement of changes in the elastic properties of concrete slabs during curing and the detection of voids [7].

Summary -- The above impact techniques offer great potential as reliable methods for flaw detection. While they appear similar in terms of the physical test procedure, different information about the test object can be obtained by using the correct instrumentation and signal processing methods. Each method is best suited for particular applications. Persons interested in using NDT methods based on stress wave propagation should develop the capability to use all the methods, so that the most appropriate one can be used for a particular situation.

These impact techniques appear to have overcome many of the difficulties that have been encountered by those who have attempted to apply existing ultrasonic pulse-echo techniques to the testing of concrete. As research continues, our understanding and the capabilities of these methods will increase, and, eventually, low-cost test systems will become available.

FUTURE DIRECTIONS

A review has been presented of recent developments in nondestructive testing of concrete. Methods for estimating in-place strength during construction and methods for flaw detection in existing structures have been discussed. Where appropriate, references have been made to existing ASTM standards. The author concludes with personal views on the future prospects for NDT of concrete.

The ultimate objective in the area of in-place strength determination is to develop a technique which is reliable, simple to use, and inexpensive. Despite many years of research, there is no method that satisfies all of these criteria. However, when properly used, existing techniques have proven to be satisfactory on actual construction projects. The author believes that additional major research efforts are not needed to develop more testing techniques to assess in-place strength *during construction*. However, there is a need to promote the proper use of available methods. The author believes that some misgivings about in-place testing exist because of poor performance resulting from misuse of these test methods rather than their inherent deficiencies. There is a need to educate potential users about the correct procedures for using existing methods. While there are ASTM methods to obtain test results, there is no standard practice in North America for planning and interpreting in-place tests. ACI Committee 228 is taking some steps in this direction, but more study is needed to understand the true reliability of the in-place strength which is estimated by using in-place testing. This requires research on actual construction projects or simulated field trials, rather than depending solely on laboratory studies.

In the area of NDT methods to estimate in-place strength in *existing construction*, additional research is warranted. The major problem in this case is how to develop the needed correlation relationship. Here, it would be desirable to have a test method which has a very robust correlation relationship, that is, one which is little affected by the actual materials present in the concrete. Methods which measure some static strength property are likely to be the best candidates for concrete strength evaluation in existing construction. This review has mentioned some potential methods, but more field performance data are needed to assess their true capabilities.

The other NDT methods that have been reviewed are those for flaw detection. This is an area where additional research is needed to develop the basis for standardization. Research programs should include the following activities:

- Development of a thorough understanding of the physics behind the test methods.
- Development of reliable computer methods for analyzing the large amount of complex data obtained by these methods.
- Development of methodology to reduce the time needed to acquire data.

Infrared thermography, radar, and stress wave propagation methods have been discussed. Among these, the author believes that stress wave propagation methods have the potential for the greatest advancements in terms of flaw detection. This is because stress wave methods are inherently sensitive to changes in density and elastic constants, which are the major characteristics which distinguish flawed from sound concrete. The major drawback to stress wave methods is that they are "point-to-point" tests, which makes the inspection of a large structure a slow process. Major technological advances are needed to permit "scanning," as is currently possible with infrared thermography, and to a lesser extent with radar. In addition, additional analytical studies are required to understand the interactions of stress waves for the various impact methods which have been discussed.

The development of reliable and relatively easy-to-use nondestructive flaw detection methods for concrete has been hindered because of difficult technical problems. These can be solved only by multi-disciplinary research teams involving experts in concrete technology, electrical and mechanical engineers, mathematicians, and computer scientists. Teams such as these have been assembled, but more are needed to meet the challenges in this exciting and evolving field.

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Table 1 Within-test coefficient of variation (COV) of break-off test

Reference	MSA (mm)	Aggregate Type	n	Range of COV (%)	Avg. COV (%)
Johansen [41]	25	?	5	Not Available	9.7
	25	RG	5		8.7
	38	?	5		12.3
	Sand		5		4.1
Keiller [42]	19	Cr	6	4.2 - 15.8	9.4
	19	RG	6		8.2
Nishikawa [66]	13	RG	10	5.1 - 13.7	9.9
	13	Cr	10		8.0
	10	RG	10		4.7
	19	RG	10		9.0
	25	RG	10		13.3
Naik et al. [61] (with inserts)	19	Cr	5,6	3.5 - 11.7	6.8
	19	RG	5,6	3.0 - 17.9	10.6
(drilled cores)	19	Cr	5	2.8 - 11.6	6.2
	19	RG	5	3.6 - 12.9	8.3
Barker & Ramirez [6]	13	RG	4	2.4 - 13.9	6.0
	13	Cr	4	2.9 - 7.2	4.8
	25	RG	4	3.8 - 14.3	6.8

MSA = Nominal maximum aggregate size
Cr = Crushed stone
RG = River gravel
n = Replicate tests

Table 2 Apparent activation energies in kJ/mol obtained from isothermal tests of concrete and mortar specimens [20]

Cementitious Materials	W/C = 0.45		W/C = 0.60	
	Concrete	Mortar	Concrete	Mortar
Type I	63.6	61.1	48.0	43.6
Type II	51.1	55.4	42.7	41.1
Type III	43.6	40.1	44.0	42.6
Type I + Fly Ash	30.0	33.1	31.2	36.6
Type I + Slag	44.7	42.7	56.0	51.3
Type I + Accelerator	44.6	54.1	50.2	52.1
Type I + Retarder	38.7	41.9	38.7	34.1

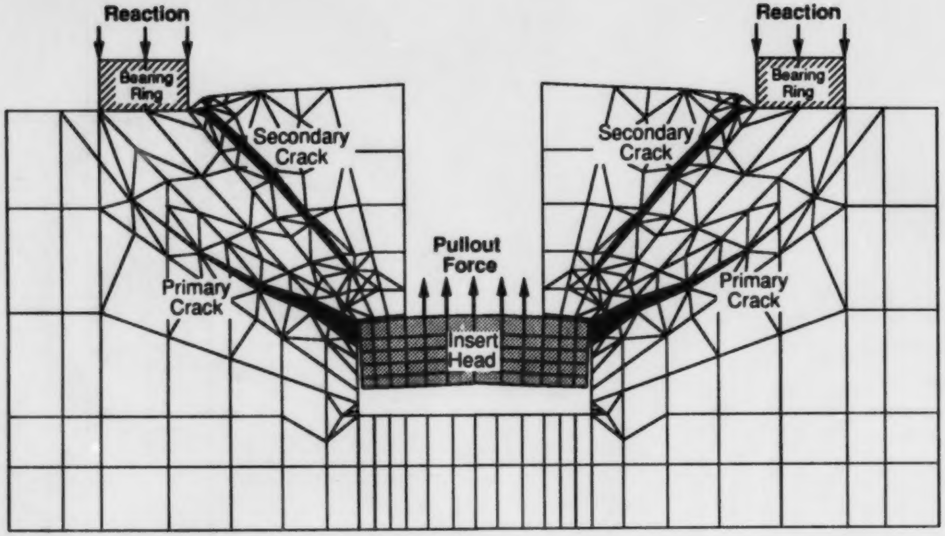

Reaction
Bearing Ring
Secondary Crack
Primary Crack
Pullout Force
Insert Head
Reaction
Bearing Ring
Secondary Crack
Primary Crack

Fig. 1 Primary and secondary circumferential crack systems obtained from a finite analysis using a discrete cracking model [38]

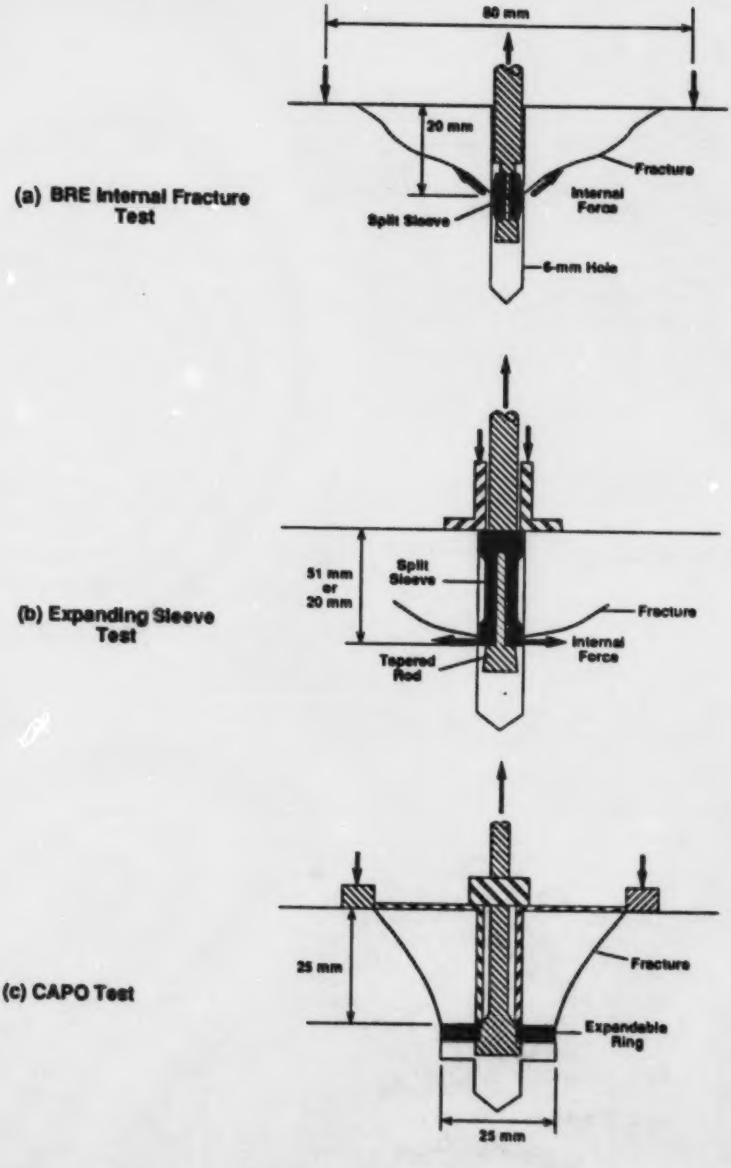
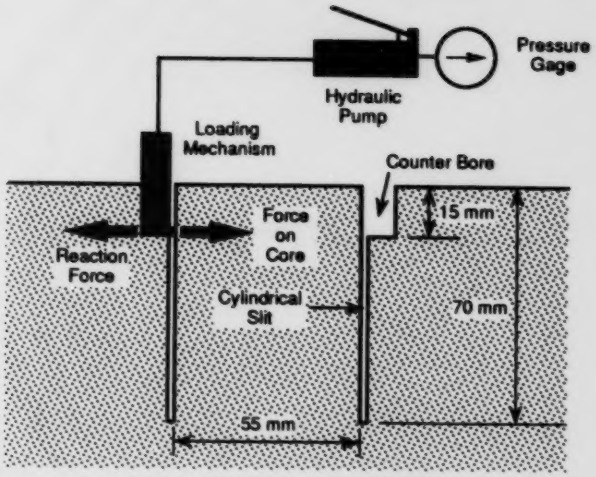
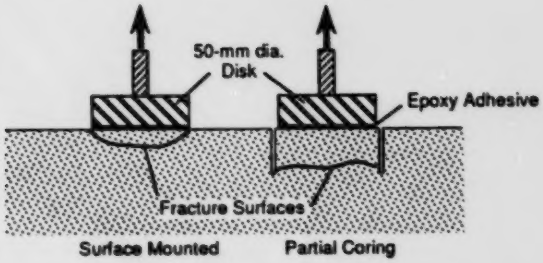

(a) BRE Internal Fracture Test
80 mm
20 mm
Fracture
Internal Force
Split Sleeve
6-mm Hole
(b) Expanding Sleeve Test
51 mm or 20 mm
Split Sleeve
Fracture
Internal Force
Tapered Rod
(c) CAPO Test
25 mm
Fracture
Expandable Ring
25 mm

Fig. 2 Examples of "post-installed" pullout test techniques: (a) BRE internal fracture test; (b) expanding sleeve technique; and (c) CAPO Test

(a) Break-off Test


Pressure Gage
Hydraulic Pump
Loading Mechanism
Counter Bore
Force on Core
Reaction Force
15 mm
Cylindrical Slit
70 mm
55 mm

(b) Pull-off Test


50-mm dia. Disk
Epoxy Adhesive
Fracture Surfaces
Surface Mounted
Partial Coring

(c) Stoll Tork Test

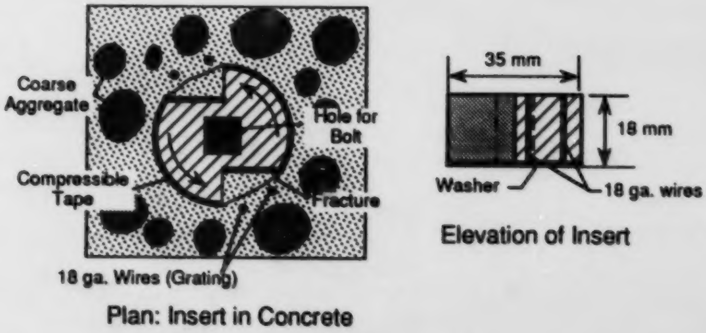

Coarse Aggregate
Hole for Bolt
Compressible Tape
Fracture
18 ga. Wires (Grating)
Plan: Insert in Concrete
35 mm
18 mm
Washer
18 ga. wires
Elevation of Insert

Fig. 3 In-place strength tests: (a) break-off test; (b) pull-off test; and (c) Stoll Tork Test

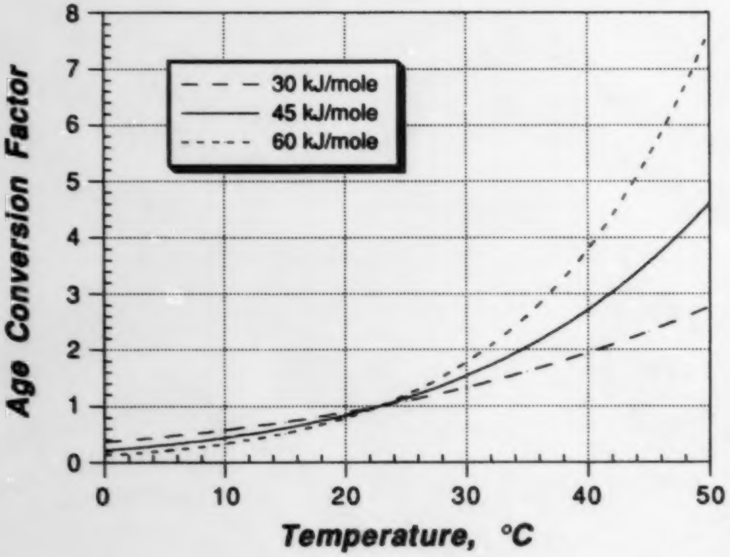


Fig. 4 Relationship between age conversion factor and curing temperature for different values of "activation energy" (reference temperature = 23 C)

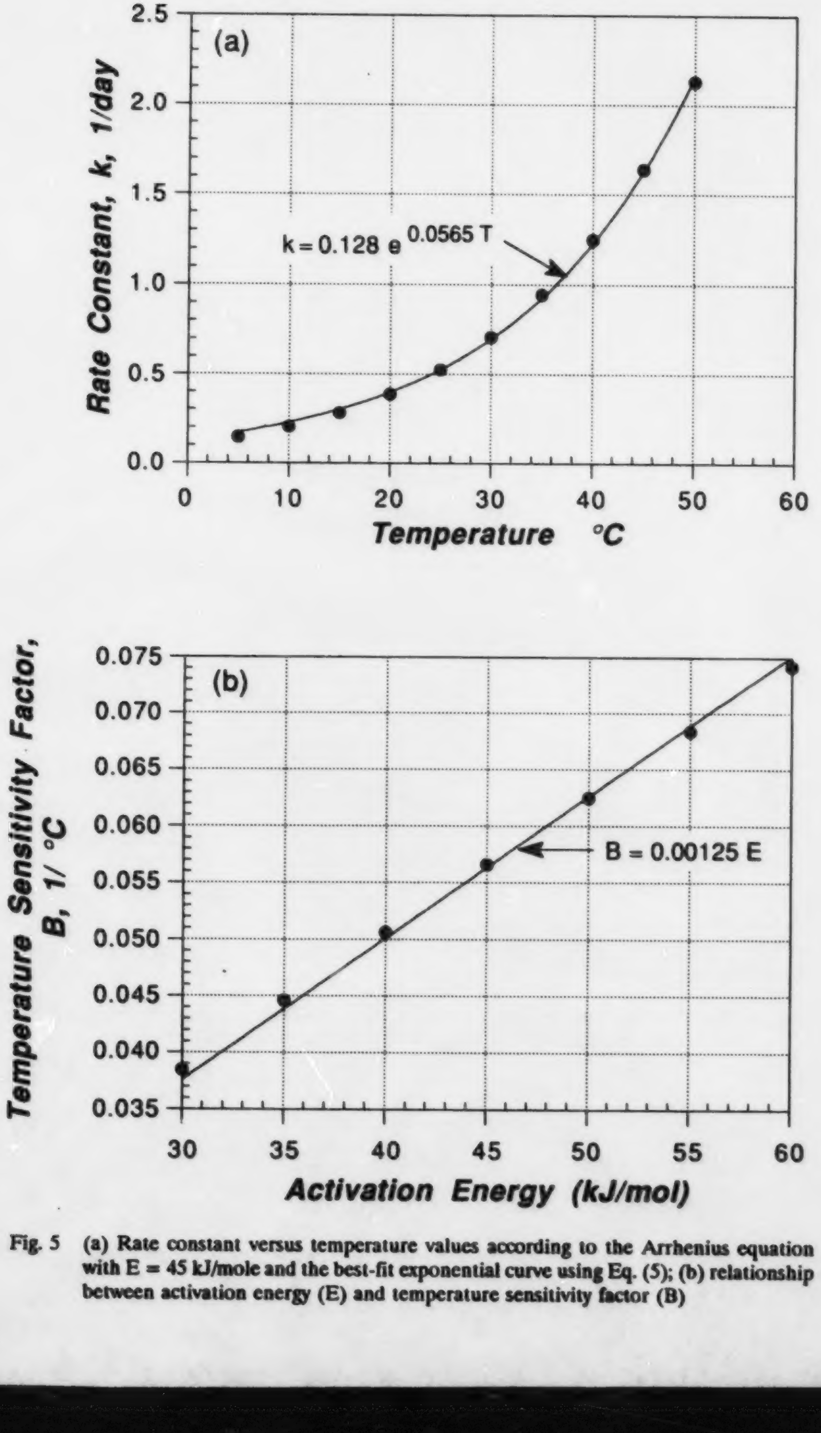


Fig. 5 (a) Rate constant versus temperature values according to the Arrhenius equation with E = 45 kJ/mole and the best-fit exponential curve using Eq. (5); (b) relationship between activation energy (E) and temperature sensitivity factor (B)

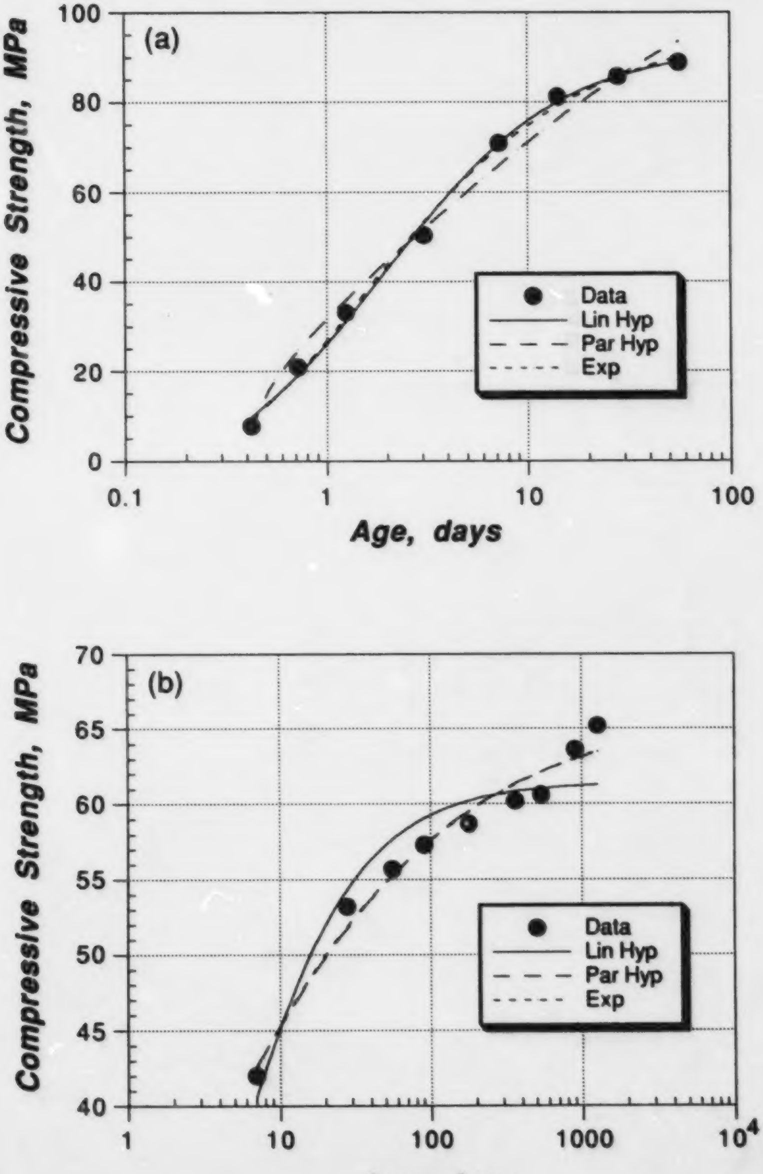


Fig. 6 Fit of hyperbolic and exponential models to strength gain data of (a) mortar cubes cured at room temperature and (b) standard-cured concrete cylinders (data from [15])

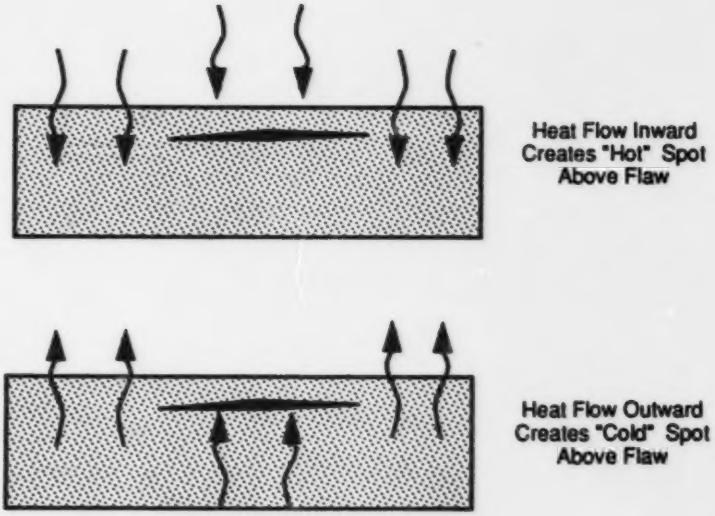
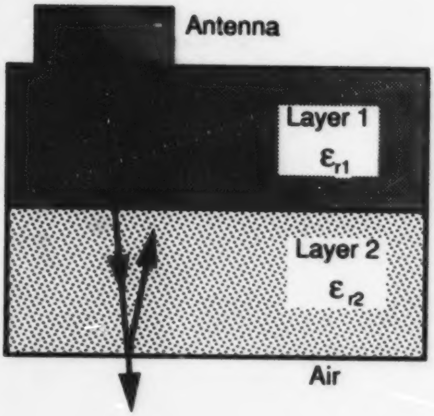

Heat Flow Inward
Creates "Hot" Spot
Above Flaw
Heat Flow Outward
Creates "Cold" Spot
Above Flaw

Fig. 7 Effect of void on heat flow through a concrete slab


Antenna
Layer 1
ε_{r1}
Layer 2
ε_{r2}
Air

Material	Range of ε_r
Air	1
P.C. Concrete*	6 to 11
Bituminous Concrete*	3 to 5
Gravel*	5 to 9
Sand*	2 to 6
Rock*	6 to 12
Water	80

* ASTM D 4748

Fig. 8(a) Reflection and transmission of radar pulse at interfaces between materials with different relative dielectric constants

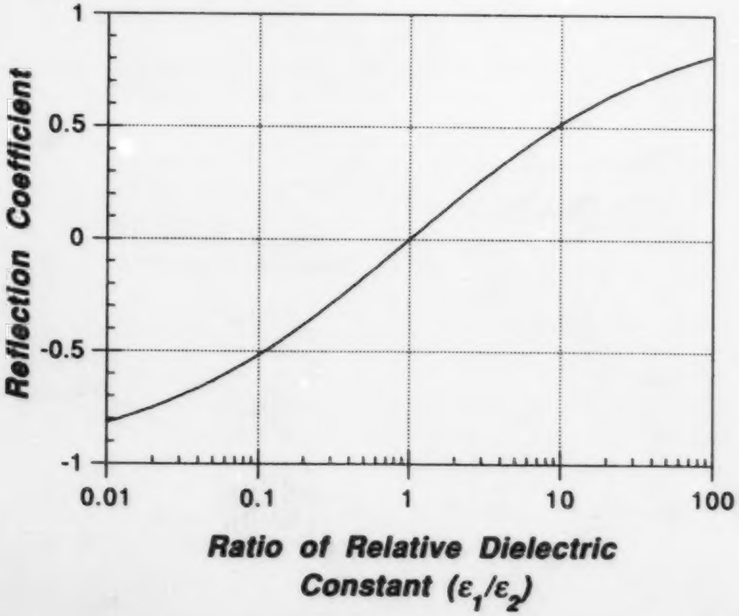


Fig. 8(b) Reflection coefficient at an interface as a function of ratio of relative dielectric constant

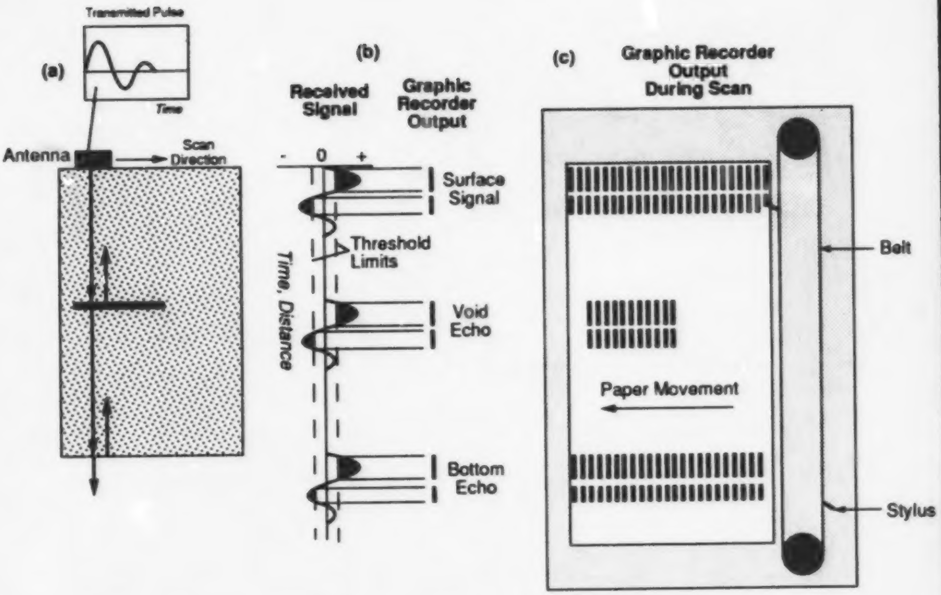

Transmitted Pulse
(a)
Time
Antenna
Scan Direction
(b)
Received Signal
Graphic Recorder Output
- 0 +
Surface Signal
Threshold Limits
Time, Distance
Void Echo
Bottom Echo
(c)
Graphic Recorder Output During Scan
Belt
Paper Movement
Stylus

Fig. 9 (a) Reflection of radar pulse within test object with void; (b) schematic of waveform from receiving antenna and threshold limits used for graphic recorder display; (c) graphic recorder display during surface scan of test object.

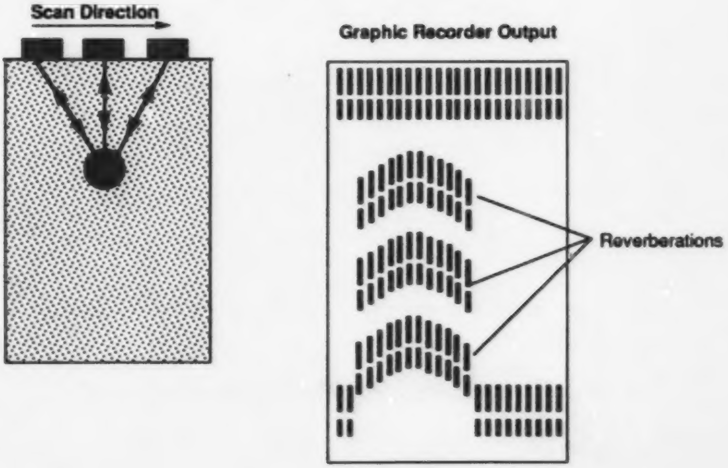

Scan Direction
Graphic Recorder Output
Reverberations

Fig. 10 Reflection of radar pulses by a reinforcing bar and the characteristic pattern on the graphic recorder

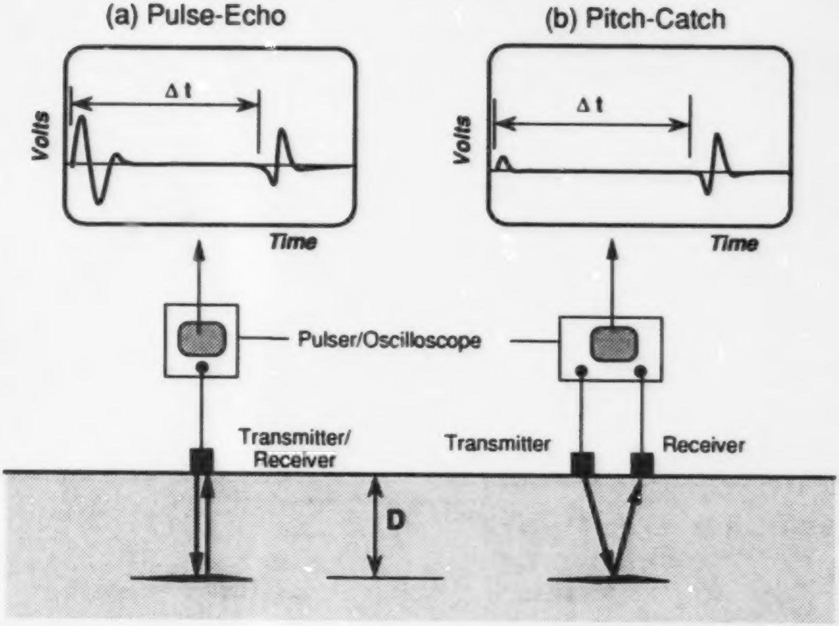

(a) Pulse-Echo
(b) Pitch-Catch
Δ t
Δ t
Volts
Volts
Time
Time
Pulser/Oscilloscope
Transmitter/
Receiver
Transmitter
Receiver
D

Fig. 11 Schematics of (a) the ultrasonic pulse-echo method and (b) the ultrasonic pitch-catch method

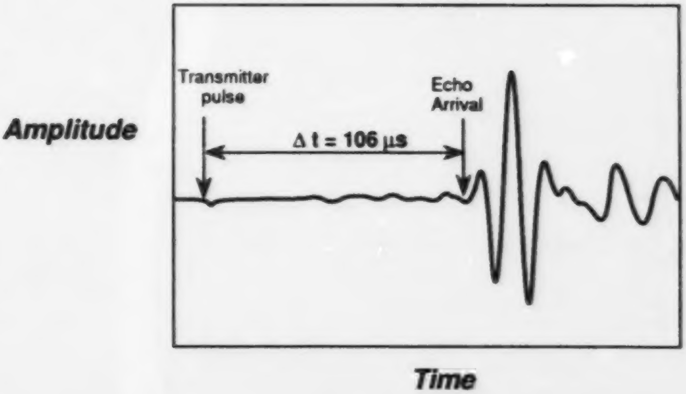


Fig. 12 Example of oscilloscope display from an ultrasonic pitch-catch test on a concrete slab [89]

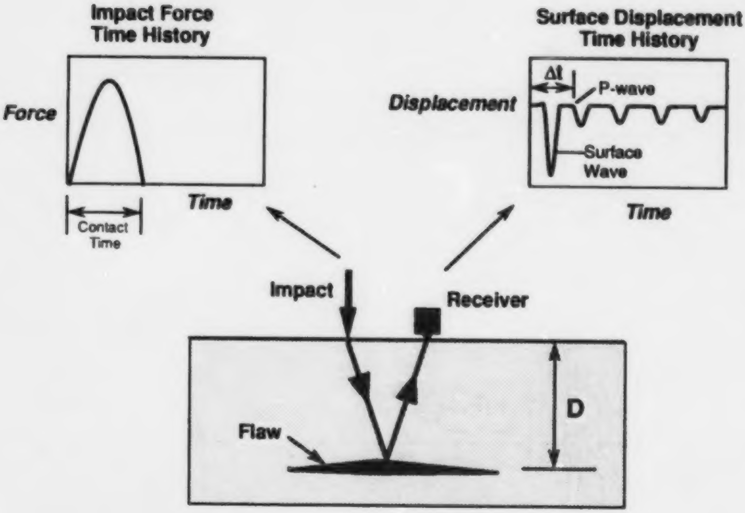

Impact Force
Time History
Force
Time
Contact
Time
Surface Displacement
Time History
Δt
P-wave
Displacement
Surface
Wave
Time
Impact
Receiver
D
Flaw

Fig. 13 Schematic of impact-echo test method

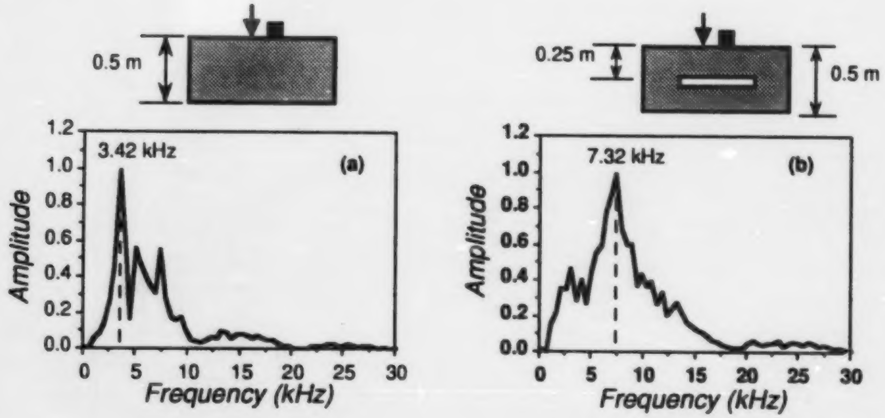


Fig. 14 Examples of impact-echo response for tests of (a) solid slab and (b) slab with a void

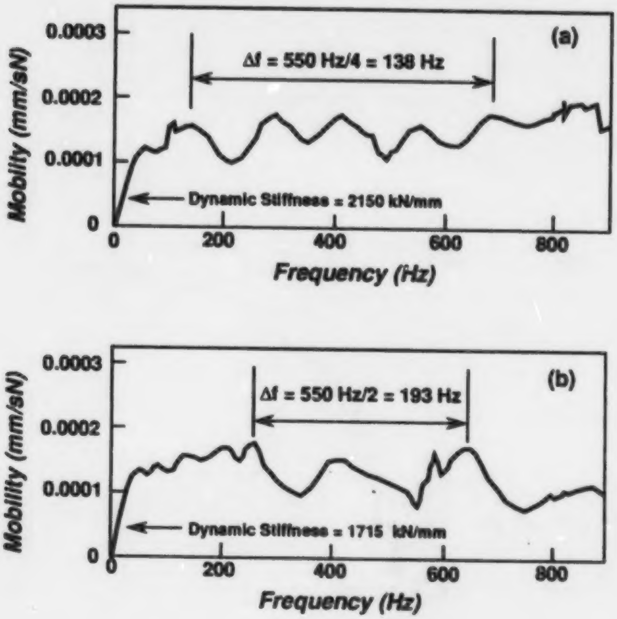


Fig. 15 Results of impulse response tests on (a) sound pile and (b) pile with full-width defect [68]

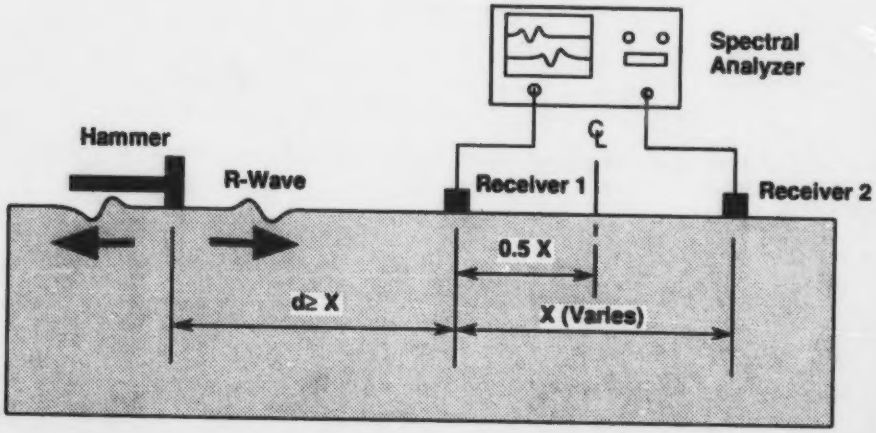

Spectral Analyzer
Hammer
R-Wave
℄
Receiver 1
Receiver 2
0.5 X
d≥ X
X (Varies)

Fig. 16 Testing configuration for the SASW method

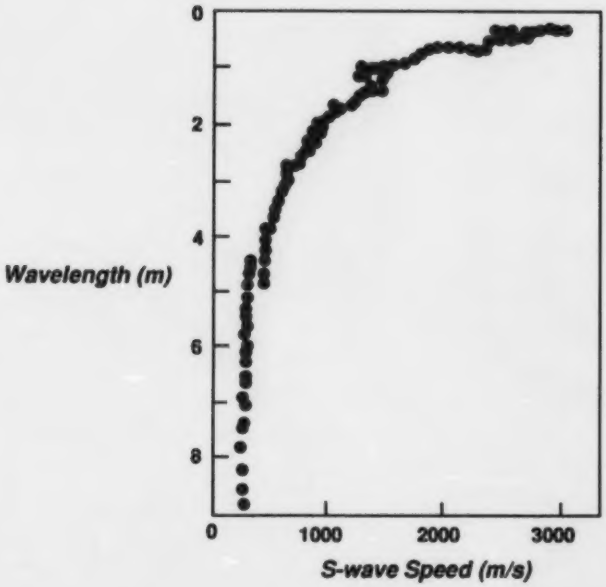


Fig. 17 Example of experimental dispersion curve from SASW test on concrete pavement [64]

Maturity of Concrete: Its Application and Limitations

By

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Synopsis: The maturity method computes maturity of the concrete as an index to predict concrete strength gain during curing. It is computed by using maturity models based upon time-temperature history. Two models, namely, the Nurse-Saul and the Arrhenius function are generally used. The Nurse-Saul function has been extensively used in determining strength gain of concrete cured in the temperature range of 10 to 32°C. Studies conducted in the last decade have indicated that Arrhenius model is valid under much wider temperature conditions relative to the Nurse-Saul function.

The maturity method has been successfully implemented in numerous construction projects to monitor strength gain. This method is preferred over others by some engineers due to its simplicity in combining the effects of fluctuating temperatures on strength development. This method can be as accurate as the method for strength determination by drilled cores if a proper relation between maturity and strength is established prior to its use. The factors such as type and source of materials, w/c, and temperature conditions to suit local conditions, etc. should be taken into account in developing this relationship. This paper reviews published studies and discusses use of the maturity method for in-situ strength measurement.

KEYWORDS: Maturity, age-strength relation, compressive strength, concrete, Model

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INTRODUCTION

Increasing cost of construction has necessitated use of accelerated construction schedules to achieve economic benefits. A knowledge of in-situ concrete strength can reduce construction time and cost by efficient movement of forms. Furthermore it also establishes safe time for formwork removal to avoid catastrophic failure of structures with consequent danger to human life.

The strength determination of concrete by testing standard cylindrical specimens is very time consuming, and it fails to take into account of variations in conditions that occur during actual placement, consolidation, and the curing of the in-situ concrete in a structure. Therefore, cylinders may not provide an estimate of the true strength gain under varying curing conditions. Consequently, attempts have been made to use in-place test methods to determine the actual rate of strength gain (1). These methods include the rebound hammer, probe penetration, pullout, ultrasonic pulse velocity, cast-in-place cylinder, maturity, etc. Most of these methods have inherent limitation in regards to their use, and interpretation of results. Recently, maturity method is gaining acceptance for predicting in-place concrete strength. This method is viewed as a simple technique which takes into account the varying effects of concrete temperature and curing time on concrete strength development. The major objective of this paper is to present the state-of-the-art information about the maturity method, and its application in determining in-place concrete strength.

MATURITY CONCEPT

Strength development in concrete occurs due to the hydration reaction between cement and water. The rate of strength development can depend upon several factors including curing conditions (temperature and age), water-to-cement ratio, type and source of cement, etc. The curing conditions are known to have the greatest effect on the rate of strength development, especially the concrete temperature (2 to 9) for a given mixture of concrete. In general, the rate of strength gain for concrete cured at high temperatures is much greater compared to lower temperatures, especially at early ages.

Since 1904, attempts have been made to evaluate the combined influence of time and temperature on the strength development characteristics of concrete (26). In the early 1950s a number of researchers proposed the combining of the effects of time and temperature by a single parameter (10,11,12). This parameter for the first time was called maturity by Saul (12). The maturity is computed as the product of time and temperature above some datum temperature following concrete casting. According to Saul (12), the datum temperature is minus 10°C. The maturity concept states that concrete specimens from a given mixture will have equal strengths at equal maturity regardless of their thermal history. This means that a unique relation exists between maturity and strength of concrete for any combination of time and temperature.

MATURITY MODELS

Maturity models are used to convert time-temperature curing history of concrete into maturity values which can be related to concrete strength gain (10,11,12,13). Numerous maturity functions have been proposed since the early 1950s. Saul (12) proposed the following relation to compute the maturity of concrete.

$$M(t,T) = \sum_{0}^{t}(T-T_0)\Delta t \qquad (1)$$

Where

M (t,T) = maturity of concrete as a function of time t and Temperature T

T = Temperature of concrete

T_0 = datum temperature, and

Δt = time interval

Eq. 1 is known as the Nurse-Saul function. The datum temperature (T_0) is the temperature at which no increase in strength of concrete occurs with time.

When comparing the two different maturity functions, it is necessary that the two functions must be compared for the same datum and reference temperatures. The reference temperature is usually taken as 20°C. Then Eq. 1 for constant temperature, T_r, can be written as

$$M(t,T) = (T_r - T_0)t_{20} \tag{2}$$

where

t_{20} = time required for reaching maturity at 20°C,

and

T_r = reference temperature.

The value of T_0 is taken as -10°C. Substituting the values T_r = 20°C and T_0 = -10°C, and using Eq. 1 and 2, the following relation can be derived.

$$t_{20} = \frac{\Sigma(T+10)\Delta t}{30} \tag{3}$$

where t_{20} is time required to reach an equivalent maturity at 20°C. This also indicates relative maturity at 20°C in hours (15).

Rastrup (13) presented a time-temperature function of the form:

$$t_1 = 2^{\frac{(T-T_r)}{10}} t_2 \tag{4}$$

where

t_1 = curing time at the temperature T_r

t_2 = the curing time at temperature T, and

T_r = reference temperature

The function proposed by Rastrup is based on well known physico-chemical rule which states the speed of reaction is doubled when the temperature is increased by 10°C. For the case of variable temperatures, a sum is formed over the time interval by the following relation (13).

$$t_{20} = \sum_0^t 2^{\frac{(T-T_r)}{10}} \Delta t \tag{5}$$

A model based on the Arrhenius function for thermal activation is widely used in some European countries and now also in North America (15). This model, as first proposed by Freiesleben-Hanson and Pedersen (14), is of the form:

$$M(t,T) = \int_0^t k \; EXP\left(\frac{-E}{RT_k}\right) dt \qquad (6)$$

where

k = a constant

T_k = temperature of concrete in degrees Kelvin

E = activation energy in kilo joules per mole, and

R = universal gas constant

The model presented in Eq. 6 has been found to be capable of taking into account the influence of temperature within a range of -10° to 80°C (15).

The hydration reactions in concrete are exothermic. The Arrhenius equation considers the activation energy (E) for the hydration process in concrete. Due to exothermic reactions in concrete, the activation energy (E) can vary with temperature. Additionally, variation in cement composition will also have an influence on the activation energy. The activation energy concept could be used to compute the values of E as shown below (14):

$$\text{At } T \geq 20^\circ C \qquad E(T) = 33.3 \qquad (7)$$

$$\text{At } T \leq 20^\circ C \qquad E(T) = 33 + 1.47(20 - T) \qquad (8)$$

where E is the activation energy E in kJ/mole.

According to the Arrhenius model (15), if temperature is constant (T = 20°C), then Eq. 6 becomes

$$M(t,20^\circ C) = k \; EXP\left(\frac{-E}{293R}\right) t_{20} \qquad (9)$$

Combining Eq. 6 and 9, t_{20} can be expressed as

$$t_{20} = \int_0^t EXP\left[\frac{E}{R}\left(\frac{1}{293} - \frac{1}{T_k}\right)\right] dt \qquad (10)$$

While Bergstrom (16), using data from literature (17,18,19) and other studies (20), showed that the Nurse-Saul maturity model is adequate to explain combined effects of temperature and time on strength development of concrete, others studies have shown that the Nurse-Saul maturity relation is invalid (21,22,23). The function developed by Rastrup is not used due to its poor accuracy relative to the Nurse-Saul model (25). Byfors (24) and Naik (15) have substantiated that the Arrhenius function is well suited for representing the combined influence of time and temperature on strength gain of concrete under a wider range of temperature conditions compared to the Nurse-Saul function.

PREVIOUS STUDIES

Malhotra (26) has extensively reviewed early studies related to the maturity method for concrete strength evaluations. However, for completeness, a brief discussion of early studies is also included in this paper.

McIntosh (10) was probably the first to develop a parameter in 1949, which he called "basic age", to combine the influence of temperature and time. The basic age was calculated as the product of time and temperature above minus 1.1°C. In this study, cube specimens were cured by using electrical curing. Based on the results obtained, he concluded that the strength of treated samples were greatly dependent upon maximum temperature (Fig. 1). To obtain a strength level, maximum temperature decreased with increasing the basic age of the specimens, and major strength gain in concrete occurred at an early age when the temperature neared the maximum.

Nurse (11), used the product of time and temperature above 0°C as a parameter to combine the effects of curing history. In this investigation, prism specimens were subjected to steam curing at atmospheric pressure and were tested for strength properties including compressive strength using various types of aggregates and cement. The test results indicated that concrete made with non-reactive aggregates (assuming no reaction between cement and aggregate) showed a non-linear relation between relative compressive strength and the product of time and temperature (Fig. 2). However, this relationship was invalid for concrete made with reactive aggregates, for which most of the strength data points were well above the minimum curve (Fig. 3).

Saul (12) carried out research work on steam curing of concrete at atmospheric pressure. He computed the maturity by Eq. 1. His equation of strength gain with maturity indicated that concrete of the same mix at the same maturity (reckoned in temperature-time) has approximately the same strength whatever combination of temperature and time go to make up that maturity. Saul reported that his relation was valid for concrete that has not reached 50°C until 1½ - 2 h, or about 100°C until 5-6 h after the time of mixing. He indicated that when concrete is raised in temperature more rapidly than above, the law of strength gain does not hold well. Under this condition, strength gain occurs more rapidly during its first few hours of treatment, afterwards, the strength was adversely affected. He further reported that the relationship is valid for the temperatures ranging between 40°C and 100°C, and times up to 28 days. Saul pointed out that concrete would not set at freezing point, but once it has set, it will continue to gain strength even at minus 10°C. He suggested a datum temperature of minus 10.5°C for long period of high and low temperatures.

In 1956, Plowman (20) attempted to develop a relationship between concrete strength and maturity. He used cube specimens that were initially subjected to normal curing for 24 hours prior to being cured at various curing temperatures. Curing temperatures varied between minus 11.5 and 18°C. Based on his test results and data derived from previous studies he developed a relation between maturity and strength as:

$$S = A + B \log(M(t,T)) \tag{11}$$

where S is strength, and A and B are empirical constants and M(t,T) is maturity based upon the Nurse-Saul function. The constants A and B are linearly related to the strength at any age. Plowman recommended a datum temperature of minus 11.7°C. He concluded that Eq. 11 was independent of the quality of cement, w/c, aggregate/cement ratio, curing temperatures below 37.8°C, and the shape of test specimens.

Several researchers including McIntosh (21), Klieger (22), and Alexander and Taplin (23) have reported that the maturity relation between strength and maturity as determined by the Nurse-Saul function, is substantially influenced by initial concrete curing temperatures. These studies pointed out that the maturity determined by Eq. 11 is not uniquely related to concrete strength when a wide variation in initial curing temperatures occurs. In accordance with the results of these studies (21,22,23), the Eq. 11 is valid only under the following conditions:

(1) The linear relation between the logarithm of maturity and strength is applicable within the range of maturity represented by 3 to 28 days at normal temperatures.

(2) The initial curing temperature of concrete is from 15.5 to 26.6°C.

(3) No loss of moisture occurs during the curing period.

Ordmand and Bondre (27) found the Plowman's strength-maturity relation, Eq. 11, valid for concrete subjected to accelerated curing at 85°C for curing cycles of 6, 19, and 23 hours with a 1/2 hour period allowed before and after heating for molding and testing of specimens.

Narayanan (28,29) indicated upper temperature for applicability of Plowman's strength maturity relation as 70°C. Beyond 70°C, the strength-maturity relation became temperature-dependent.

Nykanen (30) reported that the maturity relation determined by the Nurse-Saul model, was valid within the temperature ranging between 0 and 20°C. For temperature below 0°C, he established:

$$M(t,T) = K(T + 15)t \qquad (12)$$

where K is a constant whose value was found to vary between 0.24 to 0.4 depending upon the type of cement used. Nykanen used minus 15°C as a datum temperature as he found gain in concrete strength up to temperature levels of minus 15°C. His study indicated the dependence of maturity on the w/c as well as the quality of cement.

Ramakrishnan and Chielokitchley (31) determined compressive strength of concrete under varying w/c, aggregate/cement and curing technique. The curing methods included underwater, moist conditions, and accelerated curing. Cube specimens were subjected to curing temperatures ranging between 27 to 98°C, and the curing period varied between 1 and 91 days. In their study, strength data was found to be linearly related to logarithm of maturity. The following relations were established:

$$S_i = (1540\frac{C}{W}-950)\log M - (4325\frac{C}{W}-2680) \qquad (13)$$

$$S_m = (2050\frac{C}{W}-1375)\log M - (6250\frac{C}{W}-4385) \qquad (14)$$

where S_i and S_m compressive strength of concrete for immersion curing and humid curing, respectively. The maturity M was expressed in F-hr. Eq. 13 and 14 were found to be valid for any given maturity between 1600 to 48,000°F-hr at temperatures ranging between 21 to 100°C.

Malhotra (32) established relationship between maturity and compressive strength test data obtained from various accelerated strength tests. The maturity was determined by the Nurse-Saul model. It was found to be nonlinearly related to compressive strength (Fig.4). A similar trend was observed when the ratio of accelerated strength to 28-day compressive strength was plotted against maturity. His results showed that a separate maturity-strength relationship should be developed for each water to cement ratio and different brand of cement.

In 1971, based on test data from a number of studies, Kee (33) developed a relationship between the compressive strength (S) and age or maturity by the rectangular hyperbola of the form:

$$\frac{M(t,T)}{S} = mM(t,T) + C \qquad (15)$$

Where m and C are constants. The value of 1/m gives the maximum strength that a concrete will attain with increasing age. Plowman's Eq. 11 predicts infinite strength with infinite time which is of course, not true.

A modified form of Eq. 15 has been proposed (34,35) for better representation of experimental data at low maturity.

$$S = \frac{M-M_o}{\frac{1}{A}+\frac{(M-M_o)}{S_u}} \qquad (16)$$

where M_0 is offset maturity; strength development is assumed to begin from the maturity M_0, A = initial slope of maturity curve, and S_u is maximum compressive strength. The parameters, M_0, A, and S_u are functions of temperature.

Lew and Reichard (36) found a non-linear relation between compressive strength and maturity as:

$$S = \frac{K}{1+Ka[\log(M-30)]^{b}} \qquad (17)$$

where K, a, and b are regression coefficients. These values were found to be dependent upon cement type and water-to-cement ratio. The maturity M was computed by the Nurse-Saul function at datum temperature of minus 12.2°C. Eq. 17 was determined for concrete cured between 1.7 to 32.2°C.

Lew and Richards (37) further showed that the relation between maturity determined by the Nurse-Saul model and individual mechanical properties of concrete such as compressive strength, tensile strength, modulus of elasticity and pullout strength could be established.

Judy[1] undertook an investigation to evaluate accelerated techniques for predicting concrete compressive strength for highway construction work. Several accelerated methods were evaluated: hot water baths, low pressure steam curing, oven heating, and autogenous curing. Although these methods produced acceptable results, several deficiencies were discovered. Some procedures demanded early tests to be performed at fixed times after forming the specimens; whereas other procedures were found to be either inconvenient or required extensive equipment. Most of the above mentioned methods were found unsuitable for the conditions encountered in highway construction. The maturity technique was considered as a viable alternative to accelerated tests for in-place concrete strength determinations for highway projects.

Judy[1] tested standard cylinders, after conventional curing, without any pretreatment at 24, 48, and 72 hours. Maturity values were determined using the Nurse-Saul function. A best fit model, a modified version of Eq. 11 was developed to predict compressive of concrete.

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[1] Judy, J.M., "Potential Strength of Concrete from Early Maturity Concept", ACI Seminar, Detroit, Michigan, April 22-23, 1975.

The model is of the form:

$$S_M = S_m + b(Log\ M - Log\ m) \qquad (18)$$

where

S_M = normal compressive strength at maturity M
S_m = compressive strength at maturity m
M = degree-hour of maturity measured under standard conditions, when cured at 23°C
m = degree-hour of maturity of specimen at the time of early test (hours of age times curing temperature)
b = slope of prediction line

A plot of Eq. 18 would appear as a straight line on a semi-log paper. It differs from Eq. 11 in that the prediction line projected from the actual value of the early test, S_m, instead of the constant, A, obtained through regression, was used in Eq. 11. The difference between actual compressive strength and that predicted from Eq. 18 showed errors in the range of 15 to 760 psi. This was considered acceptable for most practical purposes.

In 1978, Naik (38) conducted experimental work to check applicability of the Nurse-Saul maturity function in determining concrete strength properties. A total of six different concrete mixtures were made. The water-to-cement ratio was varied between 0.41 and 0.54. Four of these mixtures, No. 1 through No. 4, were made with 19 mm maximum size aggregates, and Mixture No. 5 and 6 were made with 9.5 mm maximum size aggregates. The aggregate to cement ratio (A/C) by weight ranged between 5.4 and 6.5, and the concrete temperature varied between 16° and 27°C; slump varied from 75 to 125 mm; air content ranged from 0.8 to 2.2 percent; and, density of concrete was between 87 and 92 kg/m^3. A slab 0.79x0.79x0.3 m was cast for each type of concrete. For each mixture, between 50-60 standard size cylinders were cast. All specimens were covered with plastic to reduce water evaporation after casting. At an age of about 24 hours, all specimens were stripped. The cylindrical specimens were cured at 21 ± 3°C in lime-saturated water until the time of test. The test slabs from mixture No. 1 and 2 were kept inside the laboratory and the slab from mix Nos. 3 to 6 were stored outside in the open air. The maturity of both the cylindrical and slab specimens were measured using the same type of maturity meter.

The cylinders for maturity measurements were kept near the slabs in a container filled with lime-saturated water. The cylinders were immersed in water up to 6 mm below their tops, while slabs were kept moist. Fig. 5 shows the relation between maturity and the cylinder compressive strength for the various concrete mixtures investigated in this study. The results of the investigation indicated that in-place compressive strength of concrete can be predicted with about the same degree of accuracy by the maturity technique as that achieved with standard ASTM core tests (Table 1) (within plus or minus 7 percent). The author further gave data that indicated that, similar to compressive strength, other mechanical properties of concrete could also be predicted through the use of this technique. However, in order to determine these properties, relationship between the individual mechanical property and maturity needs to be determined. Naik (38) further pointed out that factors such as water to cement ratio, source and size of aggregates, source of cement, curing environment, and size of specimens could significantly affect the maturity versus strength relations.

In 1981, Volz et. al. (39) examined the strength maturity relationship for concrete subject to various combination of -1°C, 21°C and 43°C temperature regimes. The maturity was then evaluated by the Nurse-Saul model. They found that this model does not accurately account for the influence of early temperatures on compressive strength of concrete. They reported that an adjustment in the equation to account for the effects of "early" temperature is needed.

In 1983, Naik (15) performed an investigation to check the validity of the Nurse-Saul and the Arrhenius maturity models for concrete cured under winter conditions. Concrete mixtures were proportioned using water to cement ratios of 0.50, 0.60, and 0.70. Cylindrical specimens, 102x203 mm insize were used for the compressive strength measurement. These specimens were cured at nominal temperatures of 3, 13, and 23°C. The compressive strengths of test cylinders were measured at the ages of 12, 18, 24, 36, 48, 72, 120, and 168 hours. The maturity and temperature of the cylinders were also recorded at each test age. The results showed that the Arrhenius function gave a much better maturity relation with relative age (t_{20}) than the Nurse-Saul function over a wide range of concrete curing temperatures. The author recommended the use of the Arrhenius function for concrete subjected to winter as well as other curing conditions. However, it was also indicated that the Nurse-Saul function could be used for relatively higher curing temperatures, but it should not be used for winter curing conditions.

Dilly and Ledbetter (40) performed an investigation to determine the effects of curing environments on the development of relations between pullout strength, compressive strength and the maturity of concrete. In this investigation, the concrete was proportioned to have 21 MPa compressive strength obtained from a ready-mixed concrete producer. Crushed limestone, 19 mm maximum size, and a washed river run siliceous sand, were used for the coarse and fine aggregates, respectively. Concrete was cured under both controlled as well as uncontrolled/variable environment. The control environment was an air-conditioned laboratory, whereas the uncontrolled condition was the outdoor atmosphere. Cylinders were also cast for curing under controlled as well as uncontrolled conditions. A 1.8 m column was cast for curing under uncontrolled conditions. The curing temperature under controlled conditions varied between 21 to 23°C. Ambient temperature for outdoor uncontrolled environment ranged between 21 to 39°C. The maturity was computed, by measuring actual temperature from a temperature probe inserted in a cylinder and a temperature probe inserted in the column, by the Nurse-Saul function.

The analysis of the test data showed a high correlation between logarithm of maturity and compressive strength; pullout force and compressive strength; and logarithm of maturity and pullout force (Fig. 6,7,8) (40). These results revealed that maturity could be used as a technique to measure both compressive strength and pullout force within the tested range.

Recently, Parsons and Naik (41) conducted an extensive investigation concerning the use of maturity method for predicting the early age strength of concrete. Experiments were designed to study the effects of different cement types, aggregates, water to cement ratios and curing temperatures. Sixteen 100 x 200 mm cylinders were cast from each batch of concrete. Also, 150 x 300 mm cylinders were cast from each batch for 28-day compressive strength determinations.

The test specimens were cured at temperatures of 3°, 13°, 23°C and outdoor environment. Compressive strength was measured at 12, 18, 24, 36 hours, and 2, 3, 5, and 7 days. At each age, maturity was measured with a maturity meter which computed the concrete maturity using the Nurse-Saul function. Additionally, temperature and time data were also recorded to compute maturity by the Arrhenius function. Based on the analysis of test data, the authors reported that strength-maturity relationship was substantially influenced by the curing temperature, aggregate type, cement type, and water to cement ratio. In the study, datum temperature was changed from -10°C to 0°C to investigate the effects of changing the datum temperature and to establish a model for predicting the minimum concrete strength which would not be affected by different curing temperatures. A linear model between log maturity and strength was established for each level of the parameters used. The model is based on a modified maturity function evaluated using 0°C datum temperature. The constants for best fit models for various conditions are shown in Tables 2 through 5.

The analysis of relationship between compressive strength and the maturity computed using -10°C datum temperature showed lower bound of the data at 3°C. However, when the maturity computed using 0°C datum temperature, the lower bound of the data set changed from 3°C to 12°C cured concrete (41). As the datum temperature influences the lower bound of the data, an optimum datum temperature needs to be determined for estimating the minimum concrete strength for different curing temperatures. In their study, the regression line developed based upon 13°C curing and the modified maturity, given by $T_0 = 0°C$, showed a good estimate of the lower bound of compressive strength data within the experimental range.

MATURITY MEASURING INSTRUMENTS

Determination of concrete maturity values requires the knowledge of its time-temperature history. Therefore, any instrument which can record temperature of the concrete as a function of time can be used to measure maturity. The strength of in-situ concrete is then estimated using prior calibrations between maturity and the compressive strength of cylinders. For measuring concrete temperatures, sensors are embedded in concrete and connected to temperature recording/read out devices. For known time-temperature, maturity value can be computed by using maturity models. Commercially available maturity meters are appropriate to record maturity of in-place concrete (15,38). A commercially available maturity meter is depicted in Figure 9. In such meter concrete temperature is sensed by thermocouples or other sensors and maturity is automatically computed based on a maturity function, usually by the Nurse-Saul model. Maturity meters based on the Arrhenius models are also available (35). For example, a mini-maturity meter (Coma Meter) is a disposable device which automatically determines concrete maturity in accordance with the Arrhenius function[2].

Multi-channel maturity meters are also available to measure maturity values of concrete at various locations (15,35). These meters are capable of calculating maturity values using either the Nurse-Saul or the Arrhenius function as required.

CASE HISTORIES

Numerous construction projects have successfully used the maturity concept in determining the strength gain of in-situ concrete in structures during construction (42 to 48). Bickley (44) and Malhotra (42) have reported the use of maturity concept in the determination of in-situ strength of concrete during construction of the CN tower in Toronto. The maturity-strength relation was used to determine appropriate time for formwork removal. In this project, maturity-strength relation was pre-established for each concrete mixture, and was compared with the actual core test results. The maturity predictions showed a very good correlation with core test results (44). The maturity method was then used for monitoring the strength gain of the entire structure.

Mukerjee (46) also reported the use of maturity method to predict strength gain of in-place concrete in Toronto. He found that strength-maturity data could be adequately described by the Plowman's model (Eq. 11) described earlier. The constants (A and B) of this model were determined for concretes to suit local temperatures using experimentally determined data. He found that model predictions were close to the actual strength of in-place concrete determined from the push-out cylinders cast in structures. Also, the maturity method was used successfully by Mukerjee (46) to predict the in-place strength of concrete slabs during construction of buildings at the University of Waterloo in 1971 and 1972. This method was also used to monitor the strength gain of lightweight concrete floor slabs of a 37-story tower completed in Toronto to determine the earliest time for post tensioning operation of slab (46).

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[2] Hansen, A.J., "Strength Indication with a Simple Maturity Meter", Presented at the Forming Economical Concrete Buildings - An International Conference, Chicago, IL, November, 1982.

Hulshizer and Edgar (47) described a test program, involving both field and laboratory tests, to judge the performance of the maturity concept for predicting the strength gain of concrete. They reported that the maturity method was a reliable technique to evaluate in-situ concrete strength and for monitoring the actual program of curing. The concept was employed to determine safe formwork stripping times for a 10 km long, 5.8 m inside diameter, tunnel arch lining. In this work, the use of maturity concept reduced winter curing time which resulted in approximately 30% saving in heat relative to that of the conventional cold weather curing requirements. Additionally, further economic benefits resulted from reduction in labor, inspection and supervision cost, and reduced schedule durations.

CONCLUDING REMARKS

A large number of researchers have established that maturity-strength relationship can be substantially influenced by several parameters. These parameters include curing temperature, aggregate type and source, cement type and source, w/c ratio, etc.

Numerous maturity meters are commercially available to automatically determine the maturity of concrete. These meters are appropriate for monitoring the concrete strength gain in construction projects.

Due to the simplicity and ability to approximate strength gain under fluctuating temperature conditions, the maturity method has been used to monitor strength gain in many construction projects with considerable success. The use of maturity method for in-situ concrete strength determination can provide improvement in construction productivity which can result in substantial savings in energy and labor cost. In order to have an accurate prediction of strength gain in concrete, it is recommended that maturity-strength relation must be developed for this concrete prior to its use for anticipated curing conditions, for each source and type of materials, and water to cement ratio.

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Table 1. Comparison of In-Place Compressive Strength (38)

Mix No.	Actual In-Place Core Strength, psi	Predicted Equivalent Strength, psi	In-Place Cylinder	Strength Ratio, Percentage		
		From Core	From Maturity	Cyl. Str. Predicted From Core Actual Core	Cyl. Str. Predicted From Maturity Actual Core	Predicted From Maturity Predicted From Core
	4525	4935	5250	109.1	116.0	106.4
1	5045	5600	6000	111.0	118.9	107.1
	5360	6000	6100	111.9	113.8	101.7
	5425	6080	6100	112.1	112.4	100.3
	3830	4050	4300	105.7	112.3	106.2
2	4780	5250	5100	109.8	106.7	97.1
	4675	5100	5170	109.1	110.6	101.4
	4650	5075	5170	109.1	111.2	101.9
	3665	3750	3750	102.3	102.3	100.0
3	3830	3940	4200	103.9	109.7	106.6
	4200	4390	4430	104.5	105.5	100.9
	4755	5050	5030	106.2	105.8	100.4
	3125	3100	3070	99.2	98.2	99.0
4	3440	3480	3390	101.2	98.5	97.4
	3450	3490	3620	101.2	104.9	103.7
	4160	4340	4110	104.3	98.8	94.7
	2620	2790	2900	106.5	110.7	103.9
5	2955	3160	3200	106.9	108.3	101.3
	3095	3310	3480	106.9	112.4	105.1
	3715	3990	4100	107.4	110.4	102.8
	2150	2270	2400	105.6	111.6	105.7
6	2680	2850	2710	106.3	101.1	95.1
	2940	3140	3000	106.8	102.0	95.5
	3575	3840	3600	107.4	100.1	93.7

Table 2. Regression Models for 3°C Curing (41)

Cement	Aggregate	w/c	Intercept	Standard error	Slope	Standard error	R^2
Type I	Gravel	0.5	-7574	297	1666	52	0.99
		0.6	-5756	275	1207	46	0.98
		0.7	-4175	242	879	42	0.97
Type II	Gravel	0.5	-8630	509	1730	82	0.98
		0.6	-4804	383	1020	65	0.95
		0.7	-4316	543	884	92	0.88
Type I	Limestone	0.5	-7758	462	1656	76	0.98
		0.6	-8056	423	1553	67	0.98
		0.7	-3600	303	752	51	0.94
Type II	Limestone	0.5	-9529	517	1895	86	0.98
		0.6	-6838	402	1380	70	0.97
		0.7	-5127	483	988	80	0.94

Table 3. Regression Models for 13°C Curing (41)

Cement	Aggregate	w/c	Intercept	Standard error	Slope	Standard error	R^2
Type I	Gravel	0.5	-7334	196	1437	31	0.99
		0.6	-7403	201	1470	32	0.99
		0.7	-4300	105	849	17	0.99
Type II	Gravel	0.5	-7660	176	1458	27	0.99
		0.6	-5995	205	1157	33	0.98
		0.7	-4358	131	824	20	0.99
Type I	Limestone	0.5	-8391	256	1646	41	0.98
		0.6	-5663	123	1102	19	0.99
		0.7	-4723	223	892	35	0.96
Type II	Limestone	0.5	-8676	305	1639	40	0.98
		0.6	-5821	185	1088	29	0.98
		0.7	-3714	122	704	19	0.98

Table 4. Regression Models for 23°C Curing (41)

Cement	Aggregate	w/c	Intercept	Standard error	Slope	Standard error	R^2
Type I	Gravel	0.5	-8277	351	1651	51	0.98
		0.6	-7607	207	1467	30	0.99
		0.7	-5701	140	1100	20	0.99
Type II	Gravel	0.5	-8187	180	1556	26	0.99
		0.6	-6424	107	1203	15	0.99
		0.7	-4941	76	924	11	0.99
Type I	Limestone	0.5	-8175	203	1589	29	0.99
		0.6	-5439	317	1038	46	0.95
		0.7	-4323	168	829	25	0.98
Type II	Limestone	0.5	-8574	138	1595	20	0.99
		0.6	-6219	104	1147	15	0.99
		0.7	-4169	86	768	12	0.99

Table 5. Regression Models for Outdoor Temperature Curing (41)

Cement	Aggregate	w/c	Intercept	Standard error	Slope	Standard error	R^2
Type I	Gravel	0.5	-8243	174	1598	26	0.99
		0.6	-5889	174	1159	27	0.99
		0.7	-5376	135	1008	20	0.99
Type II	Gravel	0.5	-7714	159	1476	24	0.99
		0.6	-5648	235	1075	35	0.97
		0.7	-4383	137	829	21	0.98
Type I	Limestone	0.5	-7426	131	1560	21	0.99
		0.6	-6326	204	1252	31	0.99
		0.7	-4213	137	854	21	0.99
Type II	Limestone	0.5	-9179	321	1776	50	0.99
		0.6	-5199	159	1012	25	0.99
		0.7	-4168	104	791	16	0.99

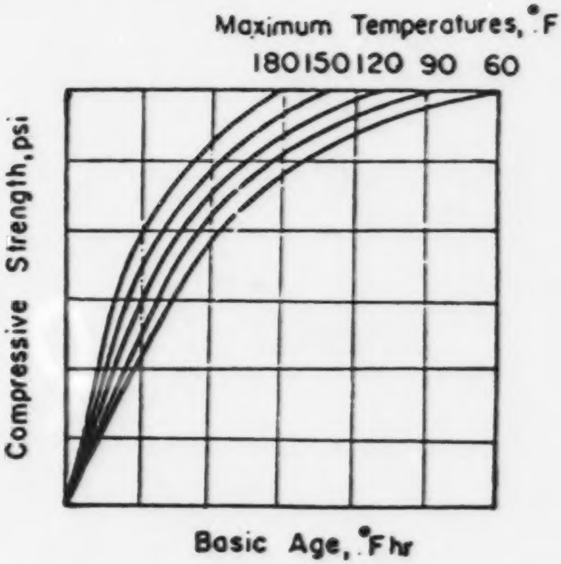


Fig. 1 - Compressive Strength Versus Basis Age Curves for Various Maximum Temperatures. From Reference 10.

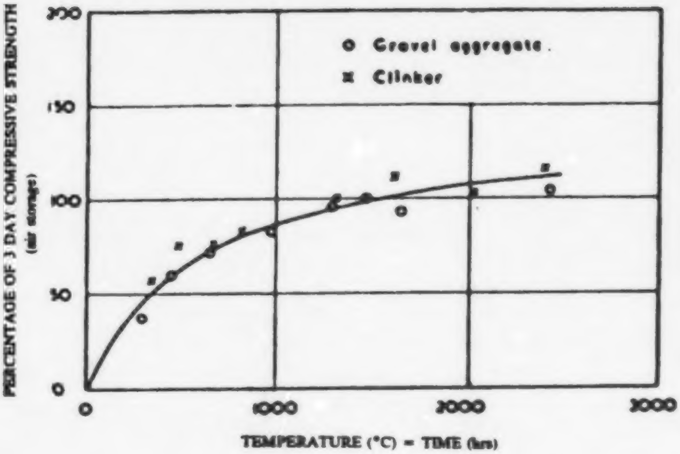


Fig. 2 - Compressive Strength Versus the Product of Time and Temperature for Nonreactive Aggregates. From Reference 11.

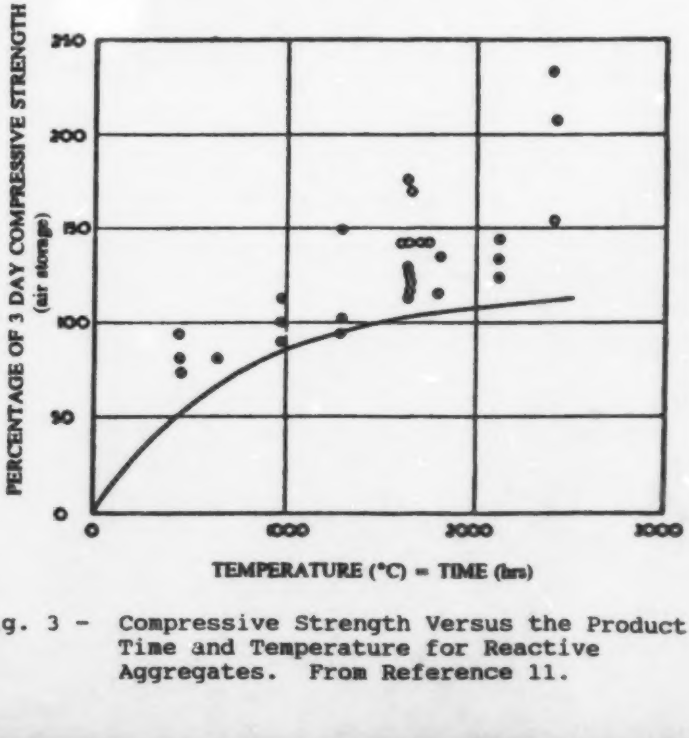


Fig. 3 - Compressive Strength Versus the Product of Time and Temperature for Reactive Aggregates. From Reference 11.

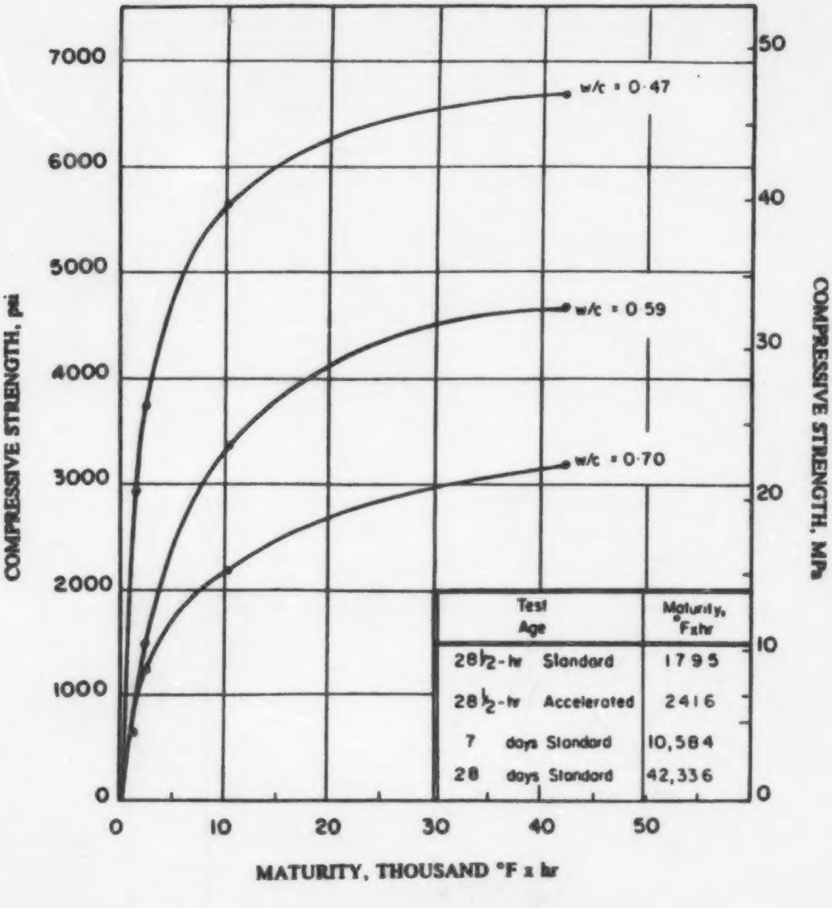
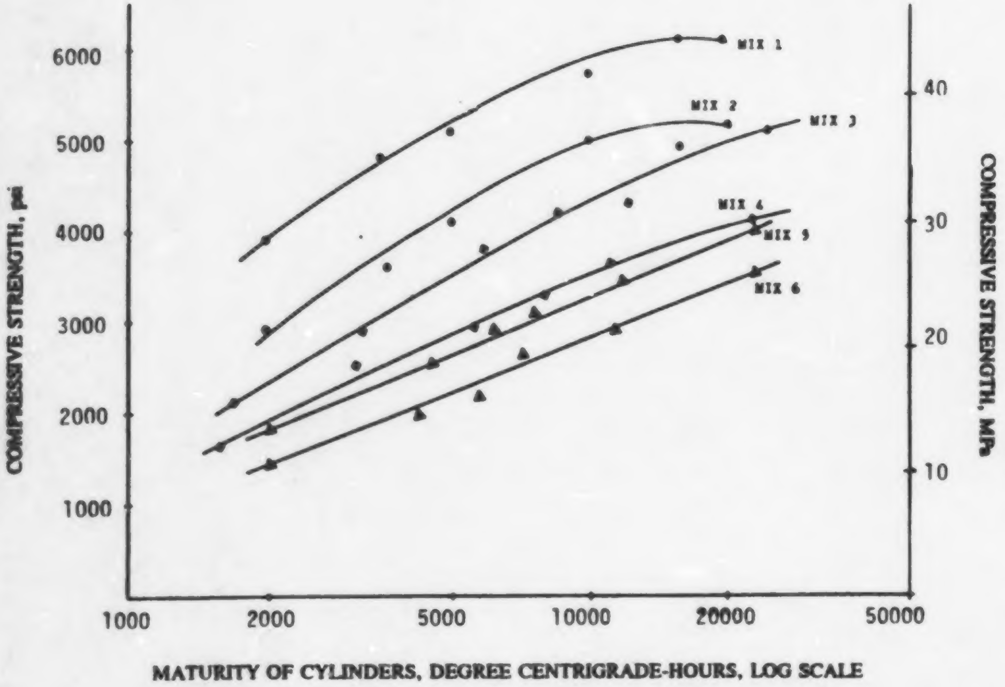


Fig. 4 - Relationship Between Maturity and Compressive Strength of Concrete. From Reference 32.



Mix No.	W/C Ratio	Maximum Aggregate Size, mm
1	0.41	19
2	0.52	19
3	0.42	19
4	0.53	19
5	0.42	9.5
6	0.54	9.5

Fig. 5 - Maturity Versus Cylinder Strength.
From Reference 38.

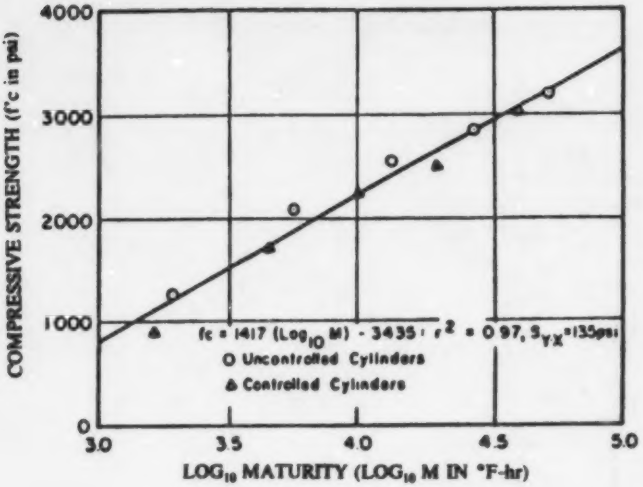


Fig. 6 - Maturity Versus Compressive Strength. From Reference 40.

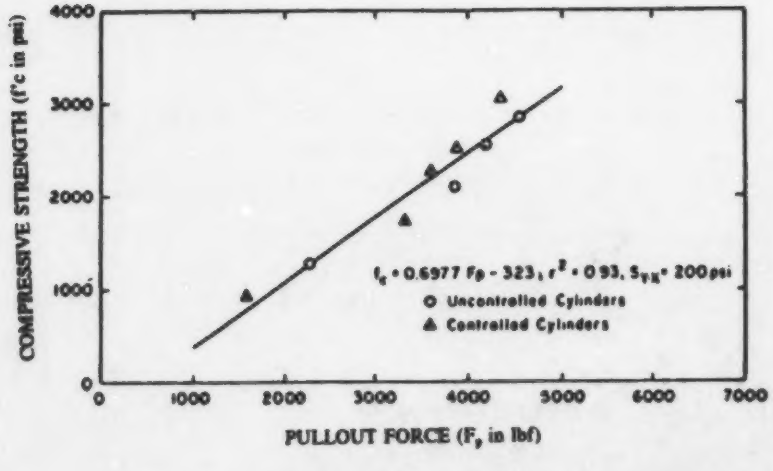


Fig. 7 - Cylinder Pullout Force Versus Cyliner Compressive Strength. From Reference 40.

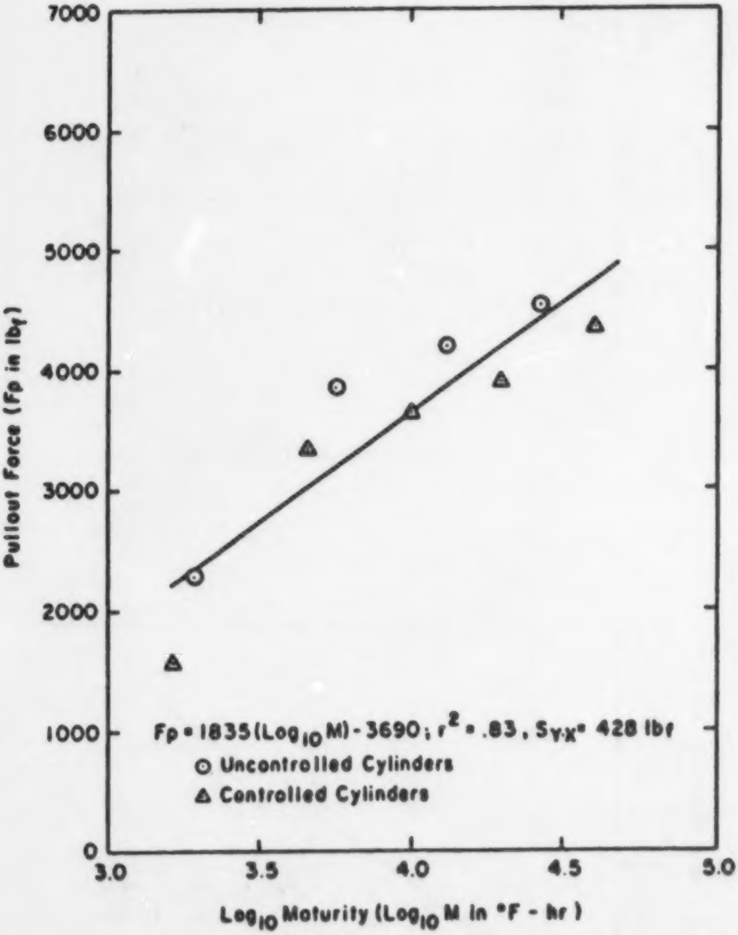


Fig. 8 - Maturity Versus Column Pullout Force. From Reference 40.

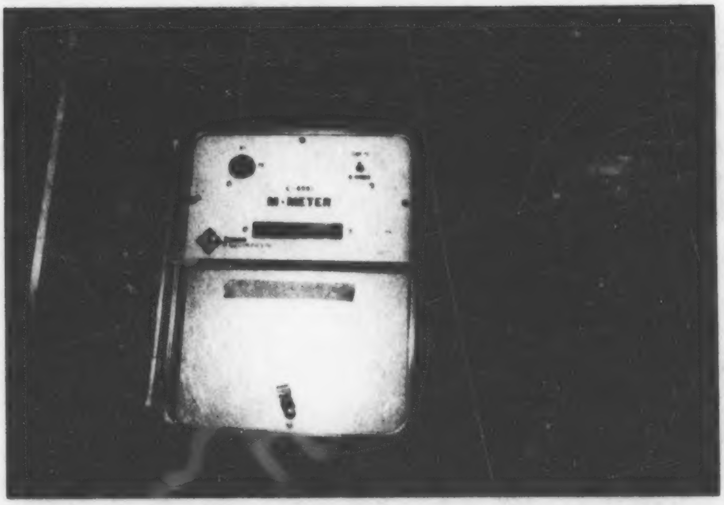


Fig. 9 - A Commercial Maturity Meter.

Roller Compacted Concrete For Dams:

The State-Of-The-Art

By Ernest K. Schrader

Synopsis: Roller compacted concrete (RCC) has rapidly evolved from a concept with laboratory studies and field trials to an accepted engineered material used throughout the world. RCC dams have many advantages. The most dramatic is significant savings in both time and money. There are different approaches to RCC dam design and construction, and wide ranges in mixture proportions. Regardless of the approach, high-speed construction and a no-slump consistency that allows the mixture to be compacted by a large vibratory roller are common factors. Poor quality aggregates and materials not meeting requirements for conventional concrete have been used successfully in RCC dams, and at times they have actually produced desirable material properties such as improved consistency at low cement contents, low elastic modulus, and better long term stress relaxation. The wide range of mixture proportions possible with RCC, its low unit cost, and the ability to develop mixtures with material properties outside the normal range of conventional concrete, provide more options than were available with conventional materials. Because construction methods and scheduling have a major impact on the quality and properties of RCC, the designer must consider the interrelationship and best combination of construction methods, schedule, materials, proportions, costs, and structural design. The contractor should understand how his activities affect the results.

Key Words: Roller Compacted Concrete, RCC, dam, strength, cost, design, RCD, joints, permeability, stability, shear, density, compaction, construction.

Ernest Schrader is a Concrete Consulting Engineer. He has been involved with the concept, planning, design or construction of approximately 50 RCC dams worldwide ranging from about 2 to 200 meters high, of which 15 are now complete.

INTRODUCTION

This state-of-the-art report concerns roller-compacted concrete (RCC) for dams. It provides an explanation of what RCC is and a summary of its development. The report covers specific technical and construction issues including aggregates and mixture proportioning, laboratory testing, and material properties, engineering and design, cost, and construction including quality control. Emphasis has been placed on areas of controversy and maximum interest such as cost, mixture proportions, watertightness, joint quality, and design options.

What is RCC?

Roller-compacted concrete (RCC) is concrete, but it is placed differently than normal industry practice and it has a different consistency. It can have a much broader range of material properties than conventionally placed concrete, it can use aggregates not meeting normal requirements, it can be placed at very high production rates, and it can be much less expensive.

By definition (1) RCC is concrete that has a consistency which allows it to be compacted with a vibratory roller. Usually a 10-ton roller intended for asphalt and granular base is used because of its compactive effort and desirable high frequency and low amplitude vibration. RCC is usually mixed in a continuous process rather than in batches, delivered with trucks or conveyors, and spread with a bulldozer in layers prior to compaction. A condensed summary of the RCC process can be found in reference (2). A more thorough summary can be found in references (1, 3, 4, 5, 6, 7).

Freshly mixed uncompacted RCC generally looks like damp gravel that might be used for a road base, although some mixtures have a wetter consistency that has more resemblance to conventional no-slump concrete. Not until the cement has reached a point near final setting or until the hydrated interior is exposed does RCC have the visual appearance of normal concrete.

Portland cement is normally the primary cementing medium, although fly ash or natural pozzolan is often used in the mixture. Slag cement has also been used. Low cement content mixtures typically use natural non-plastic fines or rock dust as a filler to compensate for the lack of paste that would otherwise exist. At times, the fines have cementing abilities.

History

The rapid world-wide acceptance of RCC dams is a result of its need, success, and economies. Twenty to thirty years ago there were occasional uses of materials that, in hindsight, can be considered as RCC. These applications were basically stabilized gravel fills that did not view the material as an engineered concrete.

In the 1960's a high-production, a no-slump mixture that could be spread with bulldozers was used at Alpe Gere Dam in Italy (8) and at Manicougan I in Canada (9). A similar process was used as late as the 1980's at Burdekin Falls Dam in Australia. These mixtures were consolidated with groups of large internal vibrators mounted on backhoes or bulldozers.

In 1970 fast construction of gravity dams using earthmoving equipment, including large rollers for compaction, was formally presented as a viable approach for the "optimum" gravity dam (10). The concept considered a section similar to, but smaller and simpler than, the section of an embankment dam. During the 1970's a number of projects ranging from laboratory and design studies, to test fills, field demonstrations, non-structural uses, and emergency mass placements, were accomplished and evaluated using RCC. These efforts (11-23), formed the basis for the first RCC dams which were constructed in the 1980's.

In the 1980's RCC dams suddenly appeared. The idea of using earthfill equipment and construction methods was refined from the "optimum" gravity dam concept to a design section more like a typical gravity dam instead of having an embankment cross section. This is reflected in publications such as "Willow Creek Dam - An Optimum RCC Dam With Vertical Upstream Face" (20), and "Willow Creek Dam - A Roller Compacted Concrete Fill" (24).

Willow Creek Dam in the United States (5, 20, 24, 25, 26, 27) and Shimajagawa Dam in Japan (26, 28, 29) are the principal structures that started the sudden acceptance of RCC dams. They are similar from the standpoint that they both used RCC, but they are very dissimilar with regard to design, purpose, construction details, size, and cost. Willow Creek was completed in 1982 and became operational in 1983. It was the first major dam designed and constructed to be essentially all RCC. Shimajagawa Dam began construction earlier and became operational later than Willow Creek, with about half of all its concrete being RCC (26).

Using RCC primarily for the interior mass of a dam is typical in Japan where a particular design and construction philosophy referred to as "Roller Compacted Dam" (RCD) is common. RCD results in an attractive dam with conventional concrete appearance and behavior, but it requires some additional cost and time compared to typical "all" RCC dams. RCD uses RCC for the interior of the dam with thick conventional mass concrete zones at the upstream and downstream faces, at the foundation, and at the top of the dam. Frequent monolith joints are used through the dam, with conventional waterstops and drains. Also typical of RCD are thick lifts with delays after the placement of each lift to allow the RCC to cure and subsequently be thoroughly cleaned prior to placing the next lift (28 - 33).

More than a dozen permanent "all" RCC dams (excluding RCD) were built in the 1980's. "All" RCC projects are generally considered to be those where the total conventional concrete volume including leveling foundations, diversions, facings, spillways, galleries, bedding, finishing the top of the dam, and other uses is on the order of about 5% or less of the RCC volume. Most of the 1980's dams are on the order of 35 to 65 meters high and have volumes on the order of 20,000 to 300,000 m^3, although the full range goes from about 7 to 90 meters with volumes of about 10,000 to 1,000,000 m^3. Summaries of typical 1980"s projects and their performance have been previously presented (4, 6, 34, 35). References for specific projects are: Willow Creek Dam (5, 20, 24, 25, 26, 27); Copperfield Dam (36, 37); Monksville Dam (38, 39, 40, 41); Upper Stillwater Dam (42, 43, 44, 45, 46, 47); Olivette Dam (48); Galesville Dam (49, 50); Middle Fork Dam (49, 51); Saco Dam (52); Urugua-I Dam (53); Kerville Dam (54); Cedar Falls Dam (55); Winchester Dam (56); dams in China (57); and dams in South Africa (58). Figures 1 through 6 show the varied size and nature of some of these dams. The concentration of these 1980's projects is in the USA, but other countries such as Australia, Mexico, China, Brazil, South Africa, and Morocco also built all RCC dams in the 1980's.

The early 1990's have continued with RCC for small and medium size dams, and they have shown extended use to much larger projects. Examples include placement of

RCC at the 100-m high Trigomil Dam and final design with scheduled completion in mid 1990's of the 140-m high La Miel II Dam (59). Serious feasibility studies for very large RCC projects have also become normal in the early 1990's. An example is the 175-m high Agos Dam with a volume of approximately 5,000,000 m^3. Large RCD projects continue to be popular in Japan.

RCC dams are no longer concentrated in the USA. They have spread throughout the world. In addition to RCD dam construction in Japan, RCC dam design and construction can now be found in USA, China, Mexico, Australia, Angola, Brazil, Argentina, Honduras, Iceland, Philippines, Morocco, Chile, Algeria, France, Italy, Spain, South Africa, Colombia, Ecuador, Greece, India, and other countries.

Advantages and Disadvantages

General

The list of advantages in RCC dam construction is extensive, but there also are some disadvantages that should be recognized. Some potential advantages can only be realized with certain types of mixtures, structural designs, production methods, weather, or other conditions. Likewise, some disadvantages only apply to certain conditions and types of mixtures. One thing that does stay constant with RCC is that each job must be thoroughly evaluated on its own. What is advantageous for one project with its set of conditions may not be advantageous for a different project, and what is a problem at one location may actually be a benefit at another location. No single design, mixture, or construction method is ideal for all projects.

Cost

The main reason for using RCC is reduced cost and time. The savings can be dramatic. RCD and more complex designs tend to have less savings on the order of about 10%, while very simple all RCC designs can have savings in excess of 50%. Each project must be evaluated on its own. A trade-off in appearance and other characteristics that may be (but not necessarily is) associated with high savings must be acceptable to the owner and compatible with technical requirements.

Although it is almost routine for efficiently designed RCC dams to be the least cost alternate when compared to other types of dams, there are conditions that may make RCC more costly in some cases. One situation where RCC may not be appropriate is when aggregate material is not reasonably available and the foundation is poor but there is an abundance of good material for impervious fill.

It is difficult to obtain final actual cost data for RCC dams, although there are an abundance of available bid price data for various portions of RCC costs, and there are several reasonable summaries of approximate overall costs (4, 35, 36). Apparent discrepancies exists in references and in costs discussed at various meetings primarily for two reasons: (1) The work and materials included in the cost may be very inclusive (mobilization, joints, engineering, facings, diversion, spillway, forming, galleries, drains, foundation preparation, etc.) or it may only include the very basic costs of RCC production (aggregate, cement, mixing, and placing). (2) Costs are sometimes based on unit bid prices which can be unbalanced and not the true prices, and which do not include subsequent added costs for claims, litigation, time extensions, modifications, and over-runs.

Because there are so many interrelated aspects of RCC, a fair way to compare the cost of different designs and projects is to consider the volumes of each design and the combined cost of: applicable mobilization; access and haul roads for RCC and related activities such as aggregate; applicable design and engineering fees; cement; pozzolan such as fly ash; RCC aggregate production; admixtures; mixing; delivery; placing; compaction; bedding mixtures; lift surface cleaning; upstream and downstream facings; watertightness; spillway; joints; detailed work at the top of the dam; foundation preparation including leveling concrete if used; galleries; and drainage. Consideration should also be given to the required quantity of excavation and the length of diversion and outlet structures.

The reason for this completeness is that some important misunderstandings have occurred without it. One example is when comparing a more massive lower strength design to a less massive higher strength design. The low-strength design will have a lower unit cost but more volume, more excavation, and longer conduits. Lower unit costs for the more massive design can result from greater volume, lower strengths, and reduced aggregate, cement, pozzolan, and admixture requirements. The high-strength design will have a higher unit cost but less volume. It may also have significantly more foundation treatment, gallery requirements, lift joint requirements, leveling concrete, and monolith joints or crack maintenance. Another example is that one design may include extra work in the RCC cost that later makes the spillway or facing less costly, whereas a different design may have very simple and less costly RCC placement but more time consuming and costly spillway construction.

Even with these problems, general cost guidelines can be provided. Typically, the final cost of RCC (all ingredients, mixing, delivery, and placing including joint treatment, drains, cure, and other directly related items) varies from about $\$23/\text{m}^3$ to $\$62/\text{m}^3$ (1991 \$US), depending on the size of the project, strength, and complexity. The following cost ranges are based on historical records of completed projects and recent estimates for future projects.

RCC Volume (m^3)	Cost ($\$/\text{m}^3$)[1]
1,000 - 5,000	\$46 - \$62
25,000 - 75,000	\$36 - \$46
200,000 - 400,000	\$26 - \$36
750,000 - 5,000,000	\$23 - \$33[2]

Notes: [1] Cost to owner based on 1991 \$US.
[2] Costs for low long dams and low cement content mixtures are generally lower than costs for high dams and higher cement content mixtures.

Historically, and in estimates of future projects, the final cost of the entire dam with all related construction such as outlets, spillways, access, foundations, facings, diversion, instrumentation, valves, finishing, and environmental restoration is about 1.5 to 2.5 times the cost of the RCC. The reasons are different from job to job, but the range stays the same. In general, the total project cost for the dam will be about twice the cost of the RCC. Basic designs with unformed faces and more massive sections tend to be less than this while more complex designs and smaller sections tend to be more than this.

Major savings of RCC dams compared to embankment dams come from incorporating the spillway into the dam rather than having a separate major excavation and structure. Additional major savings are related to the speed of construction. This often allows the dam to be raised in one dry season, thereby greatly reducing the diversion and cofferdam costs when compared to providing protection during the wet season for other designs.

Schedule

In addition to cofferdam and diversion benefits, being able to schedule RCC placement for dry seasons also improves the efficiency of construction. Fast construction reduces interest during construction and provides beneficial use (revenue) at an earlier date.

RCC can remove the dam portion of a large project with tunnels and powerhouses from the critical path, and allow a delay in the start of dam construction.

Emergency projects such as Kerville (RCC was used to rapidly build a new dam downstream of an embankment dam that was in imminent danger of failure due to overtopping (54)), and Concepcion Dam (RCC was used to rapidly build a water supply project after declaration of a national emergency (60, 61)), benefit from the speed of construction.

Equipment, Materials, and Manpower

Construction equipment needed for RCC is usually reasonably available. This includes appropriate mixers and conveyors which are the more difficult pieces of equipment to locate.

Depending on the approach taken in design and the location of the project, materials required for RCC can be difficult or very simple to procure. For example, the design of Concepcion Dam took into account the poor quality aggregate sources, cement, and pozzolan so that local materials and a low strength mixture could be used in economical construction without delays (61, 62). These materials could not meet normal requirements for conventional concrete or for RCC requiring medium to high strengths.

RCC dams can be built with high labor and low equipment requirements, but this should only be considered for smaller projects where economics and political reasons justify it, and where some seepage can be tolerated. RCC is best suited to high production with large equipment and a small labor force. Even where labor is inexpensive and readily available, experience has shown that it is best to use a small work force. It is essential that supervision understands RCC as well as dam construction for the type of construction to be used, but laborers do not require special skills other than those found in general heavy construction, supplemented by on-site orientation with RCC. It is common for contractors to use the services of a consultant to assist with portions of the work related to RCC at the start of placing.

Weather

Rainy and hot weather are arguably the biggest problems in RCC production, but they are not insurmountable. A number of major RCC projects are located in tropical areas. Hot weather causes higher internal peak temperatures, but if the environment remains warm and does not have a severe cold season, thermal cracking can usually be avoided. Hot weather may also reduce the allowable time before the concrete must be delivered, spread, and compacted, and it reduces the time that a lift surface can be exposed before it starts to suffer a significant loss of quality.

Rain is a severe problem if it is heavy or if equipment drives across the surface when it is wet. Light rain will not significantly damage a compacted surface if conveyor delivery is used so that equipment does not drive over it. Rain problems are minimized by scheduling RCC placing for the dry season, providing rain protection when appropriate, and using very high production rates with high-speed conveyor delivery.

Aggregates And Mixture Proportions

Aggregates

With respect to both grading and other properties, aggregates similar to those used in conventional concrete can be used for RCC. However, materials that would normally be considered totally unacceptable for conventional concrete have been used very successfully in RCC dam construction.

Gradings are discussed later as they relate to mixture proportions. The key to controlling segregation, minimizing cement contents, and providing a good compactable mixture begins with a grading that is more uniform and contains more material passing the 9.5-mm sieve than would be common in conventional concrete. The mixture proportioning concept that uses fines in the mixture ideally has a grading similar to an impervious gravel with non-plastic fines.

Although aggregates meeting normal concrete requirements can be used, they are not necessary in RCC dam construction. Monksville and Copperfield Dams used unwashed gravels with minimal processing that contained some friable particles of decomposed granite in the sand sizes. Middle Fork Dam used marlstone oil shale with a specific gravity of about 2.2 and absorption of 12% to 20%. Willow Creek and many other projects have beneficially used dirty overburden, or dirty quarry or gravel materials that normally would be wasted or require extensive washing. Because there were no options, Concepcion Dam (61, 62) now has the distinct honor of successfully using probably the worst quality materials in any major concrete dam.

Each project must be evaluated separately to find out how to best use the available aggregate materials. In some cases, such as where a poor foundation controls the design section, a low density may actually be beneficial. In most cases, fines are beneficial, especially if they tend to be pozzolanic, well graded, and non-plastic. Some low-strength aggregates may produce a low but tolerable strength along with a desirable high creep and strain capacity. High Los Angeles abrasion losses can usually be accepted because of service conditions, the mixing and compacting equipment, and the typical RCC grading.

RCC mixtures seem to be less affected by particle shape than conventional concrete mixtures. This is due to the mixing and delivery equipment, and the typical grading. The presence of flat and elongated particles is still undesirable, but amounts up to about 40% on any sieve size with an average below about 30% have been acceptable. Both crushed and natural rounded materials work well, but the ideal combination appears to be angular crushed coarse particles with natural rounded sand. When steep unformed slopes are to be constructed, crushed material becomes more important.

Alkali-aggregate reaction in RCC is a subject in itself. A few comments are made here. Potentially reactive aggregates have been used in RCC without problems due to different reasons including low cement contents, high pozzolan contents in some mixtures, and the use of low-alkali cements when they were available. A non-traditional approach to consider is that a slight expansion due to alkali-aggregate reaction can actually be beneficial if it offsets thermal contraction.

Mixture Designations

There are different approaches to mixture proportioning that, in turn, are related to structural design and seepage control concepts. There are no official designations, but mixtures can usually be put into one of two categories.

* High cementitious paste content mixtures with higher strengths and clean aggregates. Typically these mixtures have 125 to 250 kg/m^3 of cement plus pozzolan (C+P) with ultimate strengths on the order of about 150 to 300 kg/cm^2.

* Low cementitious paste content mixtures with lower strengths and natural fines in the aggregates. Typically these mixtures have 50 to 100 kg/m^3 of cement plus pozzolan (C+P) with ultimate strengths on the order of 50 to 150 kg/cm^2.

There can be an overlap of these categories and there can be confusion concerning the paste content with regards to whether the paste includes non-cementing fines. Wet and dry consistencies are discussed below.

Fresh Mixture Consistency

RCC mixtures have no slump. They must be stiff to support a large vibratory roller. The consistency, compactability, behavior, and properties of fresh RCC mixtures are summarized in references (1, 4, 64).

Mixtures designed by a concrete technology approach typically have a cementitious paste that will migrate to the surface and have fluidity when subjected to the combined affect of vibration and a surcharge. These mixtures may exhibit bleeding and internal pore water pressure after compaction. They may be considered to be either consolidated or compacted. Rollers and other traffic typically cause these compacted mixtures to weave or deform until the paste has hydrated sufficiently to support the wheel or drum loads.

Mixtures designed by the soils technology approach typically have no fluidity or bleeding. They will just support the roller and traffic without weaving after compaction. These mixtures are compacted rather than consolidated (64).

RCC mixtures are sometimes referred to as being wet or dry based on their appearance. This is common terminology that reasonably describes the mixture, but it does not indicate whether the actual water content is higher or lower than that of another mixture. Because of differences in aggregate (texture, shape, and grading), differences in cements and pozzolans, the effects of fines when they are included, and the influences of temperature and humidity, a mixture with a higher water content can actually be drier in appearance than another mixture with a lower water content. This is true when considering only the water above that which is absorbed by the aggregate, and it can be even more exaggerated when total water contents based on oven-dry aggregates are considered.

Chemical Admixtures

Chemical admixtures have not been effective in low cementitious content and dry RCC mixes that do not exhibit a fluid paste when subjected to vibration under full consolidation (27, 62). If enough water is added to provide a fluid paste, retarders have been useful, but this may require very high dosages on the order of two to ten times the normal rate. The retarding benefit must be balanced against the cost, complication of an additional ingredient, and any strength reduction associated with the added water needed to provide the fluid paste consistency.

Mixture Proportions

RCC mixture proportions should, and almost always do, follow the convention used for conventional concrete. That is, identifying the mass of each mixture ingredient that is contained in a compacted unit volume (cubic meter or cubic yard) of the mixture. Aggregate mass and water contents for mixture designs are based on saturated surface dry (SSD) aggregate conditions. Moisture contents are often converted to percentages based on the amount of water above SSD aggregate conditions divided by the compacted density of the total RCC mixture. Confusion has occurred when some publications and engineers have based the water content on the oven-dry aggregate mass or some other parameter. Care should be exercised when reviewing the literature, and to clarify the convention used on any project.

Some mixture proportioning methods have relied on making many mortar cubes with various cement, pozzolan, water, and admixture contents in an effort to optimize the mixture without making cylinders of the full mixture. This can be very misleading. Experience at several projects has shown that mortar cubes do not necessarily indicate how the full RCC mixture will perform. In some cases mortar cubes have actually indicated the opposite of what happened with the full RCC mixture (62).

A number of approaches have been used for RCC mixture proportioning. They basically fall into variations of two categories: (1) A more theoretical approach that tends to be based on conventional concrete technology, and (2) A less theoretical approach that is based more on soils technology and physical testing. Both approaches are well discussed in the ACI Committee 207.5R report on RCC (1) and in reference (4).

The first approach usually results in more costly but higher strength mixtures with appearance and characteristics more like conventional concrete. It often starts with the premise that the aggregates should be made from sources and with properties similar to conventional concrete aggregates. It then considers issues of conventional concrete technology such as water-cement ratio; filling the voids in the clean aggregate with paste composed of cement, water, and possibly pozzolan; and coating all of the surfaces of the aggregate particles with paste. Enough water is provided to cause a consistency measurement whereby a visibly fluid paste will migrate to the surface when the mixture is subjected to vibration and the weight of a surcharge. Detailed discussions of this mixture proportioning approach can be found in various references (1, 4, 21, 47, 63, 64, 65, 66).

The concrete technology approach to mixture proportioning has generally resulted in mixtures with high cementitious paste contents and excellent bonding properties. Often these mixtures have a very high percentage of pozzolan. At Upper Stillwater Dam, the fly ash content of some mixtures was about 75%. However, when lesser quality pozzolans or natural fines are included in the mixture, a low pozzolan content or no pozzolan may be optimum.

Some mixtures with low cement contents, and especially those with drier consistencies, have found that adding natural fines to the aggregate was just as useful as adding pozzolan. In some cases adding pozzolan was actually detrimental to the mixture, while other mixtures of similar composition have found that pozzolan contents on the order of 50% were optimum. Each project should be fully investigated to first find out what quality of cement, fly ash (pozzolan) and aggregate is available and then to find out what will produce the optimum mixture with those materials.

The second mixture proportioning approach (soils technology) starts by finding the most economical aggregate that can be produced from a local source of material. Gradings used successfully at other RCC projects serve as a guide (1, 4, 5, 27, 37, 39, 47, 62, 68). Mixtures are then made with this aggregate at different cement and water contents to establish an optimum moisture and minimum cement content that will provide the desired RCC properties. One variation of this method that uses standard soil compaction equipment for fabrication of the test specimens is suited to smaller aggregate mixtures but not to large aggregate mixtures (67).

The specific mixture procedure used to define and optimize the mixture on more RCC dams than any other method is well described for Concepcion Dam in reference (62), as well as in the ACI Committee Report 207.5R (1) and a variety of other publications (5, 27, 39, 41, 50, 53, 59, 68). It is applicable to any quality and grading of aggregates from any source, any consistency that will still support a vibratory roller, any type or content of cement and pozzolan, any or no admixtures, and any size aggregate. It uses samples of the proposed aggregates to make a series of mixtures at usually five cement contents that go from below the probable final cement content to well above it. If pozzolan is economically available, additional mixtures are made with different percentages of pozzolan but at the same total cementitious (pozzolan + cement) material content. Other mixtures may be made to verify the benefit of the fines and to help determine if extra effort to improve or increase the fines will be less expensive and technically better than adding cement or pozzolan. Usually several extra mixtures are made to determine the sensitivity of strength and compactability to minor changes in water content.

This series of mixtures is typically tested for compressive strength, density and air content, indirect (splitting cylinder) tensile strength, and modulus of elasticity at 3, 7, 14, 28, 56, 90, 180, and 365 days. It usually takes only two days to fabricate all of the samples, and the tests are simple. By having many test ages and using a semi-log time plot of the results, a very good indication of the long term properties of the concrete can be established early. Later tests confirm the indicated long-term results. By plotting the effect of different test variables, an optimum moisture, cement, fines, and pozzolan content can be selected for different structural requirements. This graphed data allows the optimum mixture to be selected for any strength requirement at any age without having to go back and make additional tests.

In some cases the results of such studies have controlled the structural and thermal design. For example, if reaching a moderate strength level can be easily accomplished with a low cement content but achieving higher strengths requires a disproportionate amount of additional cement, a design with a more massive section and lower strength is indicated.

Tests should be made on major projects using the selected optimum mixtures to determine specific heat, diffusivity, creep, coefficient of expansion, slow load strain capacity, conductivity, adiabatic temperature rise, autogenous volume change, shear strengths of lift joints subjected to different conditions (maturity, bedding, surface contamination, moisture) before placing the next layer of RCC, and the relationship between laboratory cylinders and cores of the full mixture. With a thorough basic test program and careful comparison to the properties of similar RCC mixes, the above properties can be reasonably estimated for design purposes from the series of tests, but the only way to obtain the confidence needed for a major structure has been to accomplish the tests before or during the design to verify the assumptions.

Without basic tests on initial mixture proportioning samples, estimates of the optimum mixtures and material properties, especially with low cement content mixtures, are risky. This is true with conventional concrete, and it is especially true with RCC. For example, cement efficiency factors (the strength produced per unit of cement) for mixtures with similar aggregate gradings have varied from less than 1 to over 2 kg/cm^2 for every kilogram of cement in a cubic meter of RCC. A comparison of material properties for different mixtures at selected projects is provided in the ACI report on RCC (1), reference (4), and other papers referenced for specific projects (5, 27, 39, 41, 45, 47, 50, 52, 53, 62, 64, 68, 69, 70). When comparing data, care should be taken to identify the test procedure and sample size.

When producing trial aggregates for the soils approach to mixture proportioning, economy and simplicity while still achieving a reasonable grading compared to other projects are the primary objective, but other desirable influencing factors are: keeping the nominal maximum size of the aggregate to about 75 mm for large dams and 50 mm for smaller dams; avoiding gaps in the grading; producing a slight excess of material passing each sieve smaller than the previous sieve; and producing or maintaining non plastic fines that total about 4% to 8% of the total aggregate.

Including fines has consistently allowed reductions in the cement and pozzolan content that would otherwise be higher with clean aggregate mixtures in order to fill void space, provide a compactable consistency, control segregation, and provide impermeability. Tables have been established showing the maximum permissible fines content for low cementitious material content mixtures based on the plasticity of the fines (1). As the fines become more plastic, the permissible maximum quantity decreases, and more cementitious material is needed. The source of fines has varied at different projects, ranging from material found naturally in the aggregate that normally would be washed out, to rock fines produced at a crusher or purchased as a waste product, to silt deposits added to the aggregate.

One of the complications that makes it difficult to estimate the probable optimum cement and pozzolan content of RCC without tests is the chemical and physical behavior of the fines. In some cases they are pozzolanic themselves and may render the inclusion of fly ash or other purchased pozzolan virtually useless. In other cases they may consist of material that produces soluble calcium hydroxide, in which case very high additions of fly ash can be optimum. If the fines are removed from the aggregate, these complications are removed, but, potential economic and technical benefits are also removed.

Another benefit from the inclusion of non-plastic well graded fines is physical. Without the cost and heat associated with added cementitious material, they fill void spaces thereby increasing density and compactability, increasing the paste content (if the paste is considered to include aggregate fines as well as added cementitious fines) and decreasing permeability. Reference (71) provides examples of very low permeability of these types of mixtures regardless of the cementitious material content and w/c. At Urugua-I Dam the addition of fines from non-cementitious basalt rock improved compactability and strength, and resulted in a moderate to low permeability at cement contents on the order of 60 kg/m^3. At Willow Creek Dam, the inclusion of natural silts from overburden resulted in very low permeability on the order of 10^{-5} cm/sec with cementitious material contents as low as 66 kg/m^3. If aggregate fines were not provided in these mixtures the void space would not have been filled enough by the small amount of cement to result in low permeability. The strengths of this lean concrete were on the order of 140 kg/cm^2.

Unlike the concrete technology approach to mixture proportioning which uses clean aggregate and is dependent on water-cement ratios for quality, the soils approach to mixture proportioning generally includes fines and is based on the optimum moisture content for that aggregate. This is the moisture level that provides the maximum density and usually coincides with the highest strength. This moisture level usually remains constant even if the cement content is adjusted. The water content can also be visibly found as being the maximum water content before the mixture develops internal pore pressure and begins to weave under the compactive effect of a 10-ton vibratory roller (64).

The water-cement ratio, which can be on the order of 1.0 to 2.0 for lean mixtures, is not a part of this process. References (4, 64, 71) discuss w/c in more detail. Reference (64) explains why w/c is not applicable to many RCC mixes but internal pore pressure (a soils concept) from excess water is important. This is consistent with Dr. Abrams, the originator of the water-cement ratio laws for concrete, who stipulated that water-cement ratio principles do not apply to mixtures that do not have a "workable plasticity" (72). Confusion has developed from some presentations of data for these types of mixtures showing a dramatic increase in strength due to a decrease in w/c. Unfortunately, it is not always clear that the lower w/c was usually the result of substantially increasing the cement while maintaining the same optimum water content for compaction (64, 69, 70), and the strength gain is more appropriately attributed to the increased cementitious material content.

Material Properties

Permeability has just been discussed as it relates to fines in the mixture, cementitious material content, and water content. Very low permeability in the RCC mass is typical regardless of the cementitious content if the mixture is properly proportioned with a suitable aggregate grading (71). If the mixture uses conventional concrete concepts and clean aggregate, a higher cementitious material content is needed and the w/c must be kept reasonably low in order to achieve low permeability, but a low w/c is not needed if the mixture is based on the soils approach with fines.

Misunderstandings have developed in this regard because of confusion between permeability of the RCC itself, seepage that occurs along lift joints, and other sources of seepage in completed structures. This is further complicated by the fact that seepage decreases greatly with time (4, 34, 35, 71), so it is easy to confuse a high initial seepage value for one mixture at an early age with a low seepage rate for another mixture at a later age. The data may be correct, but the lower seepage may be due to a longer service life more than to the mixture proportions.

Other confusion concerning seepage and permeability in RCC originates from sources of information that simultaneously compare permeability of drill holes or laboratory tests of cylinders and cores having no joints (or joints perpendicular to the flow path), with permeability of samples where the joints are parallel to the flow path and, at the same time, with permeability values extrapolated from measured total seepage in structures at different times. Obviously this is not a true comparison. Other influencing factors are that some permeability values extrapolated from seepage may or may not include seepage from foundation drains and other drains, seepage through transverse vertical joints or cracks that may be grouted or waterstopped, and local runoff that is part of seepage collected downstream of the structure. Also, when the permeability values are extrapolated from seepage of the structure, it is a composite value which includes the effects of placing rates, facing mixtures, enriched zones of RCC, bedding mixtures, and upstream impervious membranes.

Experience at several projects has shown that cement grouting of seeping joints does not effectively reduce the seepage rate. A very good, but often misunderstood, example is Willow Creek Dam where seepage rates immediately after grouting, both in the structure and in laboratory permeability tests of cores, were similar to the values obtained before grouting (26, 27, 71). In addition to the test data, only one location of one core showed any visible evidence of grout along a lift joint after very extensive grouting. Seepage did decrease at the project with time after grouting, but following the same pattern of reduced seepage with time experienced at other projects without grouting.

Density and Air Content of RCC depend on the specific gravity of the aggregate, the grading, the moisture content, and the degree of compaction. Fully compacted RCC will typically have less air content than conventional concrete, so its density will be slightly higher. Because RCC has been made with aggregate having a wide range of specific gravities, the specific densities of compacted RCC at different projects varies greatly from about 2.1 to 2.9.

Fully compacted RCC with a reasonable mixture proportions typically has an air content on the order of 0.5% to 1.5%. Specifications commonly require field densities to obtain an average of 98% of the maximum achievable density which is often considered to be 97% of the theoretical air-free density, providing that no density at any level within a lift of RCC is less than 93% of the theoretical air-free density. The average density is based on measurements taken at the top, middle, and bottom thirds of the lift. Air contents can be obtained in the laboratory using the pressure method for material compacted by tamping or by the vibrating table method, whichever is appropriate for the mixture.

Compressive Strengths of RCC also vary greatly depending on the aggregate, quantity and quality of fines, quantity and quality of cementitious material, degree of compaction, and moisture content. Admixtures such as retarders generally have no direct effect on strength, but indirectly they tend to decrease strengths because, in order for them to be effective, the mixture must first contain enough water to develop a paste with some fluid consistency. Even then, extremely high dosages of the admixture are required.

The chemical and physical effect of fines has already been discussed under mixture proportioning. Non-plastic fines increase the strength of low cement content mixes, but can reduce the strength of mixtures with a high cementitious material content.

Excessive water reduces the strength of RCC made with the conventional concrete technology approach to mixture proportioning because of the increased w/c. Excessive water reduces the strength of RCC made with the soils approach to mixture proportioning because the optimum moisture content for compaction is exceeded. This may be a simplification of all of the interactive phenomenon that transpire, but it explains the basic concepts.

Until about 97% compaction (based on 0% air voids) is achieved, there is a very direct and significant reduction in strength for lower compaction. However, above 97% to 98% compaction, there is no significant benefit from additional compaction, and there may even be some minor decrease in strength if the sample is subjected to excessive compactive effort (64).

The quality and composition of cementitious materials is a very important factor in the strength of RCC. It accounts for some of the reasons why one mixture at one location may achieve a strength of over 2 kg/cm^2 for every kilogram of cement in a cubic meter of RCC while a mixture with a similar grading achieves less than 1 kg/cm^2 at another location. Similarly, variations in the composition of fly ash or other pozzolan and how it reacts with the local cement and aggregate can account for some of the dramatic differences in the performance of high pozzolan content mixtures from one project to another. Due to economic, scheduling, political, importation, practical, or other reasons, the ideal cement, fly ash, or pozzolan may not be available or reasonably priced for every dam. This does not mean that RCC is not possible or that the cementitious material must be imported. It just means that allowances should be made in the design for what is practically and economically available. An example is Concepcion Dam that, besides having poor aggregates, had to use poor quality cement with even worse quality pozzolan (60, 61, 62).

Tensile Strength of RCC can vary greatly from mixture to mixture. Cementitious material content is a principal influencing factor, but moisture content and aggregate are also important. Higher cementitious material contents, lower moisture, and crushed coarse aggregate tend to increase tensile strength. However, tensile strain capacity is usually more important than the tensile strength. A highly deformable mixture with a low elastic modulus can be more desirable than a stronger mixture that is much more brittle and cracks sooner for the same amount of deformation.

The tensile strength of RCC mixtures with higher cementitious material contents and higher compressive strengths is a lower percentage of the compressive strength than occurs with low strength mixtures. Some examples of the ratio of tension to compression strength for various mixes and tests at different projects are: 4%-7% Upper Stillwater, 7%-12% Willow Creek, 9%-13% Monksville, 10%-18% Urugua-I, 12%-17% Concepcion, and 13%-19% Middle Fork.

As with conventional concrete, the user should apply a factor appropriate for the strength of the mixture when estimating direct tensile strengths from tests performed on beams or indirectly by split cylinder (Brazilian) tests.

Coefficients of Thermal Expansion for RCC mixes tend to be slightly higher than the thermal expansion coefficient of the aggregate, and just slightly less than for a conventional concrete made with the same aggregate but more cement paste.

Thermal Diffusivity and Conductivity of RCC mixes tend to be similar to the values obtained for the aggregate by itself, and similar to conventional mass concrete made with similar aggregate.

Poisson's Ratio for RCC tends be similar to values for conventional mass concrete, with typical values on the order of 0.18 to 0.24 depending on the age, aggregate, and strength.

Modulus of Elasticity and Creep for RCC can have a very broad range of values. High paste mixes and mixes made by the concrete technology approach to mixture proportioning usually produce long term values for the static and sustained elastic modulus and creep similar to conventional mass concrete; namely modulus values on the order of 200,000 to 300,000 kg/cm^2, and creep values on the order 0.03 to 0.06 Ln (time) for loading ages of about 28 to 90 days. Mixtures made with very high pozzolan contents can be expected to have lower early age values of static modulus but higher later age values.

Low cementitious material content mixtures (less than about 85 kg/m^3) can have very low elastic modulus values and high creep rates. The decrease in modulus and increase in creep tends to be exponentially proportional to the decreases in strength below about 100 kg/cm^2. Each incremental decrease in cement content has much more effect than the previous incremental decrease. Each mixture and aggregate performs differently. Static modulus values on the order of

10,000 to 110,000 kg/cm^2 are reasonable for cementitious material contents on the order of 60 to 75 kg/m^3 at ages of 3 to 90 days. Corresponding ultimate values could be on the order of 60,000 to 180,000 kg/cm^2. Because of the high creep rates possible with these mixtures, the sustained modulus values (73) are usually about 5% to 20% lower than the static modulus. Creep values on the order of about 0.10 to 1.50 Ln (time) are reasonable for loading ages of 28 to 90 days.

At lower strength levels (below about 70 kg/cm^2) and at early ages, the ratio of static modulus to compressive strength has typically varied between about 800 and 3000, but at later ages and for higher strengths it tends to be more constant at about 900 to 1400.

Tensile Strain Capacity is related to the modulus of elasticity and tensile strength. Fast load strain capacities (maximum deformability without apparent cracking) are obtained with the load applied over a time span of seconds or minutes. However, in dams, the primary concern is for slow load strain capacity where the load from external loading and internal thermal cooling develops over a long period of time. Because of creep, a dam can usually undergo more deformation without cracking if it is slowly stretched verses when it is suddenly deformed. Therefore mixtures with a low elastic modulus during the period of loading and with a high creep rate, such as occurs with low cementitious material content RCC mixtures, can have good slow load strain properties despite low strengths. References (1, 22, 38, 62, 73, 74,) discuss this concept further.
Lower strength mixtures tend to have a much higher ratio of tensile strength to compressive strength. In addition, the lower cement contents cause less thermal stress to develop. All of these factors combine to make it possible for the strain capacity to remain stable or increase with a decrease in cementitious paste content and strength. The ultimate example is a mixture with no cement which would have infinite strain capacity. It would not be strong enough to crack. It would just deform. Of course this is not a practical mixture because it must have enough strength to resist the applied loads and environmental conditions.

Typical slow-load strain capacities for RCC dam mixes are on the order of about 90 to 150 microstrains, but values outside of this range are possible. Each mixture should be evaluated, if not by direct slow-load strain capacity tests, then indirectly through extrapolation from static modulus of elasticity and creep studies. The problem with extrapolation is that it assumes the creep rate in tension will be similar to compression (as tested). This can be conservative, especially for mixtures that contain a relatively large amount of coarse aggregate. Aggregate to aggregate contact can decrease creep in compression.

Thermal Stress Coefficients establish how much internal tensile stress will develop in a given mixture for each degree of temperature drop over a specified period of time, assuming the RCC is restrained. RCC can have a broad range of values depending on the mixture proportions and aggregate. It is influenced by creep, coefficient of thermal expansion, and the elastic modulus. Stronger mixtures typically have much higher stress coefficients. For a time period of 28 to 365 days typical values for low strength mixtures (60 to 130 kg/cm^2) with low cementitious material contents are on the order of 0.5 to 0.8 kg/cm2 per C-degree. Higher strength mixtures (150 to 300 kg/cm^2) have stress coefficients on the order of 1.0 to 1.8 kg/cm^2 per C-degree.

Autogenous Volume Change (the increase or decrease in size with no applied load or environmental change) cannot be reliably estimated in conventional mass concrete without at least some test data. This is true of RCC also, and especially with mixtures that have a peculiar cement, pozzolan, and aggregate. It is not unusual to have early expansion followed by a later period of contraction, or visa versa.

Adiabatic Temperature Rise based on calculations of the heat of hydration of the cement and properties of the aggregates have been reasonably accurate for some RCC mixes and very inaccurate for others. There does not appear to be any good indicator of when they will be reliable. Testing the actual full mixture is the only way to be sure. Companion tests of hydration of the heat of the cement by itself and the heat of hydration of the cement with pozzolan (if used) are useful for later reference.

Mixtures with lower cement contents tend to produce higher temperature rises per unit of cement. This is even more evident if pozzolan is included. A comparison of the 28-day adiabatic rise for various RCC mixtures made with ASTM type II cement with the moderate heat-of-hydration requirement shows temperature rises ranging from about 0.16 to 0.30 C degrees per kilogram of cement in each cubic meter of RCC.

Cavitation and Erosion Resistance of RCC has been surprisingly good, but experience is still somewhat limited. Reference (22) contains a summary of the performance of different RCC mixtures to different cavitation and water erosion tests. Based on these limited data, which included high velocity and high head full scale trials for a limited duration as well as laboratory and other tests, cavitation and erosion rates have been developed and used with caution to justify exposed RCC spillway surfaces that are subjected to infrequent use at Willow Creek and Galesville dams. Assuming good quality mixtures with cementitious material contents on the order of 100 to 175 kg/m^3, an erosion rate of 0.01 kg/m^2 was extrapolated for a rolled surface, and 0.24 kg/m^2 for a rough surface. Experience at low head dams that have been overtopped confirm these numbers as being reasonable or conservative. High head spillway surfaces of RCC have been built but not yet used.

Resistance to Freezing and Thawing of RCC exposed at the downstream face of dams has been very good despite the fact that in laboratory tests these materials performed badly. This includes projects such as Monksville, Winchester, and Willow Creek. These projects have unformed and uncompacted downstream faces of exposed RCC with cementitious material contents of the exposed face of 62, 104, and 151 kg/m^3 respectively. Each project receives almost daily cycles of freezing and thawing during much of the winter. Monksville and Willow Creek have saturation from lift joint seepage. None of these dams have shown any change in the downstream face with time. Other projects in severe climates with temperatures that can reach -30 C and which used about 300 mm of conventional concrete facing have shown no distress.

Based on test data and observations the following deterioration rates are typical of what has been used to estimate the effects of freezing and thawing for design and planning purposes on some RCC dams. These estimates are for low cementitious material contents in the range of 50 to 150 kg/m^3. The rate for unformed and uncompacted surfaces subjected to freeze-thaw cycles that penetrate about 25 mm is 4 to 10 cycles per mm of erosion. Compacted interior RCC is estimated to deteriorate at about 40 to 80 cycles per mm of erosion.

Shear strength of an unjointed RCC mass, RCC containing joints with strengths similar to the mass, and RCC with the load normal to the joints will generally be 10% to 20% of the compressive strength for strengths in excess of 200 kg/cm^2, about 15% to 25% for strengths on the order of 150 kg/cm^2, and about 25% to 30% for strengths on the order of 70 kg/cm^2.

The strength of RCC lift joints is dependent on the consistency of the concrete, aggregate characteristics, degree of compaction at the lift surface, cementitious material content, maturity of the lift surface when it is covered with the next lift, condition of the lift surface, and lift surface treatments. A wide range of possible joint shear strengths exists depending on these variables with extremes from about 0 to 20 kg/cm^2 for cohesion and 30 to 60 degrees for friction angles.

One summary of strengths for 35 tests including 5 projects, 10 mixtures, 8 joint conditions, and 5 aggregate types resulted in an average cohesion of 11 kg/cm^2 and a friction angle of 50 degrees (70). Each project should be carefully examined and preferably tested to determine the shear capacity of its joints. Well proportioned dry low cementitious material content mixtures with reasonable aggregates and construction controls can be expected to produce friction angles on the order of 40 to 45 degrees, with cohesion values on the order of 3 to 8 kg/cm^2. High paste wet mixtures typically have similar or slightly better friction angles and cohesion values that may range from about 8 to 20 kg/cm^2.

Substantial testing of lift joints has been performed on various projects with very useful results reported for those mixtures and conditions (1, 4, 5, 7, 27, 43, 53, 62, 66, 69, 70, 75, 76). Care must be exercised not to use one publication or one set of results that is based on one set of conditions at one project as an absolute basis for what will occur at another project. A general idea can be developed based on a compilation of information from other projects and a knowledge of the mixture and materials proposed for a new project, but absolute values should come from testing of the specific mixture and conditions in question.

Every job may have its own peculiarities, but generalized observations typical of most RCC follow: Shear strength increase with age. A wetter consistency at the same cementitious material content can slightly decrease the cohesion value if the moisture is above optimum, and it can increase it if it is below optimum. After a joint exposure maturity of about 500 C degree hours, cohesion decreases rapidly. Slight surface contamination or damage caused by haul vehicles driving on compacted surfaces, greatly decreases cohesion. For a given aggregate and mixture, adding cement or pozzolan provides little improvement in shear unless the amount is large. Both a mortar bedding and a well sanded conventional concrete bedding mixture between RCC layers greatly improve cohesion and slightly improve friction.

DESIGN

General

Efficient RCC dam designs are a balance of structural considerations including dam stability, stresses, and foundation stability; thermal stresses; construction methods and rates of placement; and mixture proportions including their material properties. Because of the low cost of RCC, the wide range of material properties possible with RCC (including properties not possible with conventional concrete), and the ability to use marginal or normally unsuitable materials if necessary, there are many more possibilities for the designer to evaluate when considering RCC than there would be for a conventional dam. Consequently the designer has a more complex task, but he also has more opportunity to save time and money for the project while maximizing the use of available resources and materials.

Structural design of RCC dams uses the same basic criteria and procedures used for conventional concrete dams. After the RCC has hardened in place, it is concrete. The concrete was just placed by a more efficient method than had historically been used, and it may have a design section and material properties that are outside the normal range of conventionally placed concrete structures. These attributes, plus the rapid rate of placement and the minimization or elimination of monolith joints, may require a more in-depth study of the structure. Discussions of structural design of RCC dams can be found in references (1, 3, 4, 5, 7, 37, 42, 43, 53, 59, 61, 69, 70, 77, 78).

The primary areas of extra attention in RCC dams are: overdesign strength requirements for variability; design section options; upstream and downstream facings including the spillway; lift joint quality (tension, shear, and watertightness); and thermal stresses.

Overdesign strength requirements for variability are required by most codes for general conventional concrete construction. Sometimes the overdesign is achieved through an arbitrary extra factor of safety applied to the required design strength, but more appropriately specific statistical procedures such as those outlined by ASTM or ACI are used. It has not been common to apply these specific procedures to dam construction, although various methods have been employed historically to provide a reasonable overdesign in dams. This overdesign is an increase in the average strength requirement to account for the fact that not all cylinders will test at exactly the design strength level. Some will be higher and some will probably be lower. The overdesign factor is a method of limiting the probable number of low strengths to acceptable levels.

Special care is needed with the application of overdesign factors in mass concrete, including RCC, because extra cement used to increase the average strength may result in an unacceptable increase in thermal stresses. Also, in high production RCC where the results of meaningful cylinder tests are not available until the structure is essentially complete, testing of variability in the fresh mixture as placed has developed as a practical method of statistical control rather than using cylinder strengths. The overdesign requirement and its application to RCC dams is discussed in detail in reference (78).

Design section and facing options for large and small RCC dams are shown in Figures 7 and 8. The same options could technically be applied to conventional concrete, but the cost and construction methods used in conventional construction make most of them impractical. Generalized comments on these design options follow.

As with conventional mass concrete, RCC dams can be curved or straight. There is no technical limit on the possible height of RCC dams (1, 2). They can be designed as gravity sections using two or three-dimensional analyses. Complex three-dimensional analyses of gravity sections are usually only used for special situations and large structures. Arch dams and gravity-arch dams can be designed with RCC, although attention should be given to construction space.

Very low dams, cofferdams, and emergency structures lend themselves to design and construction using a steep sloped embankment section, Figure 7. Aprons are useful. These sections can be built on gravel and pervious foundations if necessary, the Ferris Diversion Dam on the North Platte River, Wyoming, USA, being a good example.

The following discussion of concepts identified in brackets [] refer to Figure 7.

Typical dam sections [1] are not zoned for different mixtures unless the dam has a height in excess of about 60 m. Experience has shown that zones of different strengths between given elevations are very easy to construct, but zoning in the upstream to downstream direction can be a practical problem that is usually avoided.

The top of the dam [2] can be overtopped. Freeboard is not necessary, and a parapet wall can be considered for wave run-up. The top width of the dam [3] must consider the cost of RCC added for construction or operational convenience, recognizing that RCC in this part of the dam is slower and more expensive than at the lower internal mass areas. Added mass in this area seldom has much structural benefit. The response of the mass at the top of the dam to seismic loads and the influence of its height should be evaluated.

The upstream face of the dam can be vertical or inclined. Facing methods [16] are discussed later. Adding the upstream piece [4] will shift the resultant force and widen the base for increased sliding stability where it may be needed in higher dams.

Galleries [5] should be kept to a minimum, and should use designs that are compatible with RCC delivery and placing methods. Unnecessary galleries, especially at the narrow top portion of dams, slow construction and reduce the overall quality of the in-place concrete. Crowning the gallery is simple to accomplish with the "uncemented temporary fill" method of gallery construction. Rock drains [6] can be a good substitute for drainage galleries in many situations. Internal drain holes [5C] can be easily and quickly drilled with rotary percussion equipment. Typically these are 76 mm in diameter at about 3-to-4 m spacings.

A fillet [7] at the top of the dam can reduce the possibility of cracking due to inertia of the top section of the dam when subjected to ground acceleration. It also can help shift the resultant force towards the upstream face and reduce tensile stress at the bottom lifts of a medium to high dam. An added benefit is the extra room for construction. A fillet at the toe of the dam [8] adds sliding stability and reduces maximum compressive stresses. This can be helpful in a high dam with higher stresses than can be achieved with local materials and economical mixes, and also for poor foundation conditions. A key [9] made with a high paste mixture or a lean mixture with bedding between the RCC layers to provide the necessary shear strength may be necessary in the lower portion of dams with poor foundation stability. An added section of RCC with a parabolic shaped downstream face can be used to reduce compressive stresses and increase sliding stability so that a balance exists between the available strength and the stresses in the foundation and mass of the dam. Changing the slope of the downstream face is not practical with conventional dam forming and construction methods but it is simple with RCC.

Other practical options for improving foundation stability on poor foundations that would not support a normal concrete dam section include a stepped massive toe key [11] which provides added support [12] and [13], and a rock or embankment berm [14] which adds the vertical force [15] as well as reducing the sliding resistance required in the lower RCC lifts. The berm may reduce tensile stresses that could develop across lift joints at the toe in the lower part of high dams in the construction condition (reservoir empty).

An upstream wall [17] in the lower part of the dam should be considered for construction reasons where the valley sides are steep. The area upstream of the wall can be filled with excavation waste to provide a desirable work platform. Through planned drainage, it also can control upstream seepage into the foundation area and help keep it clean. The wall normally is not part of the dam, but whether it is or not, special attention is needed at the contact of the wall and the upstream face of the dam with the foundation [17A].

Shear strength, watertightness, and tensile strength of RCC layer to layer interfaces are important and sometimes controversial issues. These interrelated issues have received considerable attention on virtually every dam. They combine construction methods and production rates with design concepts and mixture proportions. Specific discussions, some of which may seem to be in conflict, can be found in more detail in references (1, 3, 4, 5, 7, 17, 37, 39, 40, 42, 43, 45, 52, 53, 58, 61, 62, 69, 70, 71).

Mixtures with high cementitious material contents and wetter consistencies typically achieve better cohesion, tensile strength, and watertightness than mixtures with a drier consistency and low cementitious material content. Friction factors for both mixtures are similar. Designs, especially with medium to high dams, that use the drier low cementitious paste content mixtures require more mass and special attention to control seepage and to improve sliding resistance at the base of the dam.

The material property aspects of joint shear strengths were discussed earlier. When additional joint strength is required for added stability within a dam, the dam can be widened, a mixture with a much higher paste can be used, or a bedding mixture can be used between the RCC layers for a portion of the width of the dam as necessary to develop the sliding resistance on each layer [15A]. A bedding mixture is especially useful when cold joints with reduced shear capacity develop.

Tensile strength for structural stability in an uncracked section is a controversial issue amongst dam designers, with a majority appearing to favor designs that have no tension except for thermal stresses for the normal reservoir condition. Transient tensile stress for the seismic conditions are not unusual. However, the upstream zone near the base of practical design sections for high dams often requires carrying some tension, for the normal reservoir condition. Again, there is disagreement but most designers prefer to limit this tension to some very safe level on the order of 1% to 2% of the compressive strength. Without bedding mixture at the lift joints, the direct static tensile strength of dry RCC mixtures typically varies from 0% to about 5% of the compressive strength, with a reliable average on the order of 2%. High cementitious paste mixtures, and dry lean mixtures with bedding can have static direct tensile strengths ranging from 2% to over 10% of the compressive strength with a reliable average on the order of about 6%. Tests should be made to verify whatever strength is assumed for any given design. Transient sudden tensile strengths of concrete can be 50% higher than the static strength.

Watertightness, permeability, and seepage have been discussed earlier as they relate to mixture proportions and material properties. The joints of untreated dry lean mixtures can be expected to seep. The level of seepage will be low and probably of no technical consequence, but it will be visible and special procedures will be necessary if seepage control is required.

Using a high paste content mixture is one method to relative joint seepage but cracking may then become a problem due to heat and a high elastic modulus with low creep (42, 73). Providing very fast construction on the order of one lift every few hours and retarding the set time is another option, but it is seldom practical. Other design options for watertightness and seepage control, and their performance, are discussed in a number of the references, the principal ones being (1, 4, 34, 35, 56, 59, 61, 71). These methods fall into three categories. First, providing a bedding mixture between the RCC layers from the upstream face to a minimum distance downstream that is equal to at least 8% of the hydraulic head at that level. Second, providing an upstream impervious facing such as a continuous PVC membrane that is either exposed or behind precast concrete facing panels, or waterstopped large conventional concrete slabs similar to the facing on a rockfill dam. Third, providing a conventional concrete facing placed with the RCC, which has performed poorly when tried.

Various options for both upstream and downstream surfaces are shown in Figure 8. They provide appearances ranging from tolerable to excellent, and allow RCC production to proceed at a fast rate. General comments follow. Conventional reinforced concrete slabs [16A] extend the duration of the project and can be costly. An unreinforced facing slab [16B] placed just behind the RCC would require special admixtures and mix proportions to avoid cracking that would allow seepage without reinforcing. Facings placed as the RCC layers proceed [16C, 16D, 16E] are techniques from the early development of RCC. They can be attractive, but they have little effect on seepage and they can have frequent cracks (about every 1 to 2 m) unless a special mixture with high-range water reducer is used. They almost always are unreinforced. Vertical joints with effective waterstops have been virtually impossible to construct with these systems. If the thickness of the facing is greater than about 300 mm, the facing can develop thermal cracks and make placement of the RCC more difficult. To overcome this, most projects have used thin facings while others have used a wet RCC mixture and temperature controls in the facing mixture. Even then, cores of the RCC near the facing have not always been satisfactory.

Precast panels [16F] have been an effective facing method that continues to gain popularity. A membrane of polyvinyl chloride or high density polyethylene [16G] has provided very good leakage reduction when used behind precast panels or without panels. A very economical and fast way of providing a technically acceptable but unattractive face with significant seepage reduction is with a small (0 to 10 cm thick) amount of bedding mixture at the formed face which then carries downstream between RCC layers [16H].

Downstream facings [18A through 18E] are mostly variations of techniques used at the upstream face, except that steps can be easily built using forms that are moved without machinery, and large seepage reduction is not needed or desired. Steps made of conventional concrete are typical in spillways, although unformed faces and smooth conventional concrete faces are also used.

Thermal stress in RCC dams is a major subject which is well discussed in the references (22, 27, 34, 35, 38, 43, 45, 46, 52, 60, 73, 74). Thermal properties including the important thermal stress coefficient have discussed in the section on material properties. Some general comments follow:

Important thermal aspects of RCC are: (1) The cement contents can be extraordinarily low in lean mixtures and mixtures with high pozzolan contents, resulting in much less heat than is typical in conventional mass concrete; (2) RCC dams often have few contraction joints or no joints, which results in more lateral restraint than is typical in normal mass placements; and (3) High paste content mixtures, even if the paste contains a high pozzolan content, typically have elastic and creep properties that may make them 2 to 10 times more brittle than lean mixtures, thereby resulting in much higher thermal stresses for the same amount of temperature change.

It is not practical or economical to provide forced pre-cooling or post cooling of RCC. However, requiring the production of aggregates in cold winter months and storing them in large stockpiles until they are used in warmer months has resulted in reduced placing temperatures. Also, the rate of placing, hours of placing (night vs day), and schedule for starting and finishing placing are critical factors to the thermal stress analysis. The designer must pay much more attention to the details of construction and schedule than with conventional concrete. Some conventional concrete thermal wisdom does not apply to RCC. For example, a low cement content mixture may have an early adiabatic temperature rise that is less than the heat absorbed from the sun by a thin RCC layer with a large area. In this case, placing at a faster rate can result in lower temperatures - exactly opposite of what would occur with normal placing of conventional mass concrete.

CONSTRUCTION

Because of the interrelationship between mixture proportions, structural design, material properties, and construction, many aspects of construction have already been covered. It is important for the contractor to understand that construction equipment and methods have a direct effect on design, and that some procedures, production rates, and equipment may not provide the quality or characteristics needed for a particular design. What was acceptable for one design and set of conditions may not be acceptable for a different design or set of conditions. Construction is discussed in general in references (1, 2, 3, 4, 6, 7, 17, 64, 71, 81). References for specific projects discuss methods and problems for those particular dams.

Rapid construction is important to quality and simplicity, as well as profit. Common production capacities are on the order of 300 m^3/hr for medium sized dams and over 700 m^3/hr for large dams. It is more difficult to do a good job of quality construction at low production capacity than at high production rates. In general, RCC mixtures that do not contain a retarder should be delivered within 10 minutes of when mixing started, spread within 10 minutes of when they are delivered, and compacted within 10 minutes of when they are spread. Typical specifications require that in no case should the elapsed time from the start of mixing until the end of compaction exceed 40 minutes. The exposed edge of RCC lifts should also be compacted or advanced within about 40 minutes of when the material was mixed. These time limits are good guides, but consideration should be given to the particular characteristics of each mixture, the temperature, the location and criticality of the mixture at the location in question, and the practicality of construction.

Mixing is typically done by continuous mixers with specially designed volumetrically controlled pugmills. Some projects have also used batch-type pugmills. Reference (81) deals specifically with mixing and delivery of RCC, with detailed comments concerning various mixing equipment and procedures. Other references for specific projects discuss the equipment at each job. Mixers not specifically designed for RCC mass mixtures usually perform poorly, even if they worked well with other materials. High maintenance and poor mixing are the biggest problems. Accurately introducing the concrete ingredients into the mixer is only one part of the process. Uniformly distributing the materials throughout the mixture, especially when the mixture has a small amount of cement or pozzolan and when it contains natural fines, is the other part of the mixing process (78, 81). It is actually the quality of the mixture at the point of compaction that is important, so the effects of added mixing or of segregation and drying during delivery and spreading should also be considered.

Delivery is discussed in many references. Reference (81) covers trucks, scrapers, chutes, hoppers, loaders, and conveyors. Trucks and scrapers were the primary methods of delivery for most of the 1980's projects. Experience showed that this system works, but it is much more difficult to properly achieve without adversely damaging the RCC lift surfaces than is apparent. Cores and evaluation of completed structures as well as observations during construction have proven this to be common. The three main problems with haul vehicles are: segregation of the mixture as it is dumped into and out of the haul unit; continually raising the haul roads as fast as the RCC is raised without disturbing production; and, damage to lift surfaces by the haul vehicles.

Partial conveyor systems that deliver from the mixer to a fixed location on the dam have solved part of the problem, but they still leave the critical issue of damage to the lift surfaces from traffic. This problem is particularly difficult when rain or excess watering from the curing process exists. Even slight damage or contamination can result in a very major decrease in joint sliding stability and tensile bond, and increased seepage - enough to seriously compromise the integrity of the dam. Constant cleaning, re-cleaning, delays, and arguments are common. Dry lean mixtures do not bleed, so they can most efficiently be cleaned by blowing the dry or SSD surface with air. Wet mixtures that bleed are less likely to have contamination from loose sandy particles typical of the dry lean mixtures, but they can develop laitance from bleed water and from higher paste contents. This is similar to the problem of lift joint cleaning in conventional mass concrete, and it is better handled by water washing.

Total conveyor delivery from the mixer to the final location on the dam has essentially solved the problems of reliable delivery without contamination. It is especially useful in steep canyons, rainy environments, and large projects. The equipment should be specifically designed for RCC of the type to be placed, and for continuous delivery of mass concrete at high speeds with clean return belt surfaces (81).

RCC is typically spread with bulldozers and compacted with 10-ton vibratory rollers. Vibration caused by the dozers as well as the pressure of the tracks provides considerable pre-compaction before the roller is used. Some specifications require a minimum number of passes of the dozer for this purpose. Typical lift thicknesses are 30 to 40 cm for dry lean mixtures. Wet mixtures are often placed in a series of layers that may be on the order of 150 to 400 mm thick until a total thickness of 0.5 to 1.0 m is reached, at which time the composite lift is rolled. The thickness used at each project should be based on test results that verify uniform compaction of the full lift, and which assure that compaction within the allowed time limits is achieved. Density requirements have been discussed previously.

Vibratory rollers can be single or double drum. Usually six passes after spreading by the bulldozer is adequate although thicker lifts and very dry mixtures may require additional passes. Most mixtures achieve nearly full compaction after only four passes. Over compaction can improve densities at the bottom of the lift but will reduce density at the top of the lift. Compaction at abutment contacts and in small areas where the large rollers can not operate can be obtained with high energy walk-behind tampers and vibratory rollers (64).

After compaction, the surface of exposed layers must be cured as with conventional concrete. This usually is accomplished with water trucks and sprinkler systems that mist the surface. Harsh water jets and typical road-type watering nozzles will damage the surface. Curing should be continuous until the next layer of RCC is placed. The surface can be allowed to dry to a saturated surface-dry (SSD) condition or just slightly drier immediately prior to placing the next lift in order to improve cleaning by air jets.

Various forming and facing systems were discussed earlier. Form systems need special attention. It is not unusual in steep canyons and at the top of most dams for the rate of forming to control the rate of RCC placement. Precast panels help, but do not solve all of the problems. Slip formed facing elements are more suited to long and large dams than to small dams where the equipment space requirements, delivery of concrete, and quality control required for the concrete may be problems. Some designers have had serious concerns about cracking in the facing, bond to the RCC, and the strength of the facing when the RCC is placed against it, but experience has shown that these problems can be overcome with proper equipment, designs, and inspection.

From a construction standpoint on projects that require watertightness, upstream-facing systems that use low permeability barriers such as a continuous sheet of PVC or HDPE result in much easier construction because the issue of joint quality is greatly relaxed. The joints still need to have adequate shear strength, but this is relatively easy to achieve without the close inspection needed if the design required low leakage or high tensile strength across lift joints.

Quality control during construction is almost entirely geared towards assuring that equipment, materials, procedures, and timing are correct at the time of the work so that the end product will be acceptable. Cylinders are tested for record purposes, but by the time meaningful results are received the dam may be essentially complete. Even tests at the mixing plant have limited value as a control. For example, at a rate of 700 m^3/hr it is too late if it takes an hour to obtain a sample, test it, and react to an unsatisfactory result. Most specifications are geared towards performing tests during one shift of operation, reacting to the results if necessary during the next shift, and assuring that the corrective action was successful during the third shift. Tests of the fresh mixture include moisture content, cement content, compacted density, unit weight, unit weight of the air-free mortar and aggregate distribution (78). If these are correct, the mixture has the potential to achieve its full strength. Accelerated curing of mortar cubes can provide a quick indicator of cement quality.

Tests at the placement typically consist of assuring that the specified time limits for mixing, delivery, spreading, and compaction are met. Density and moisture tests are continually made using a double probe nuclear gage that should be calibrated against various density blocks of the freshly compacted RCC.

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Figure 1. Willow Creek Dam

Figure 2. Urugua-I Dam

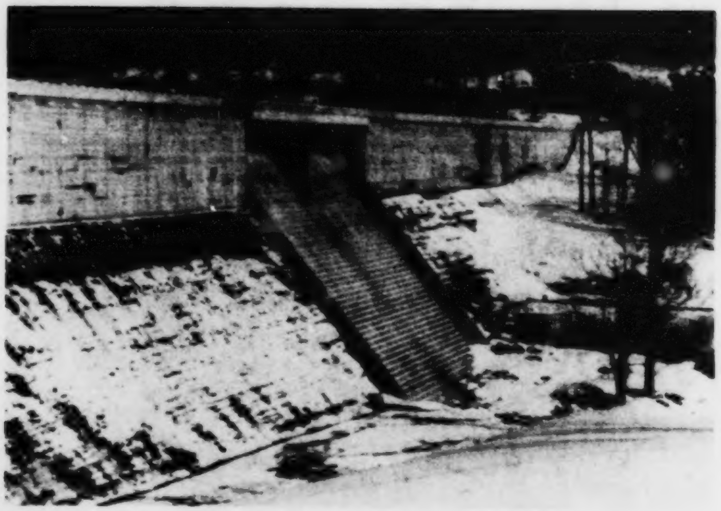


Figure 3. Monksville Dam

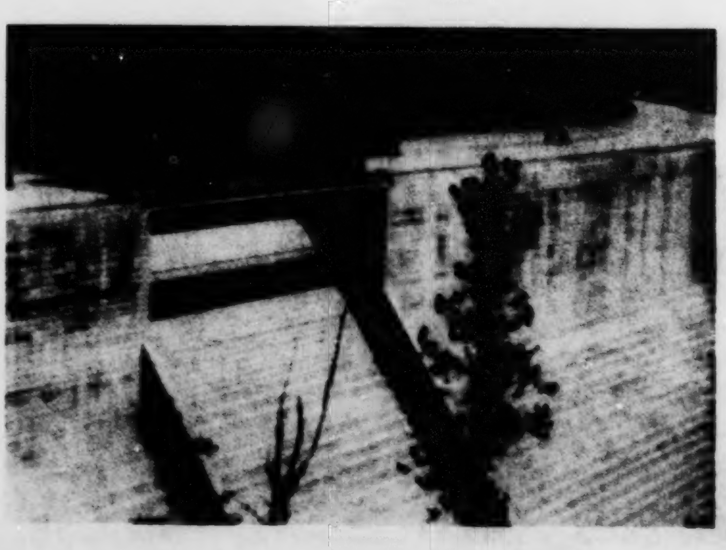


Figure 4. Stagecoach Dam

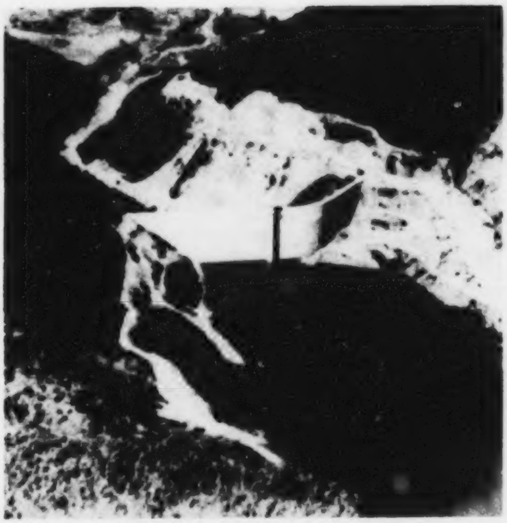
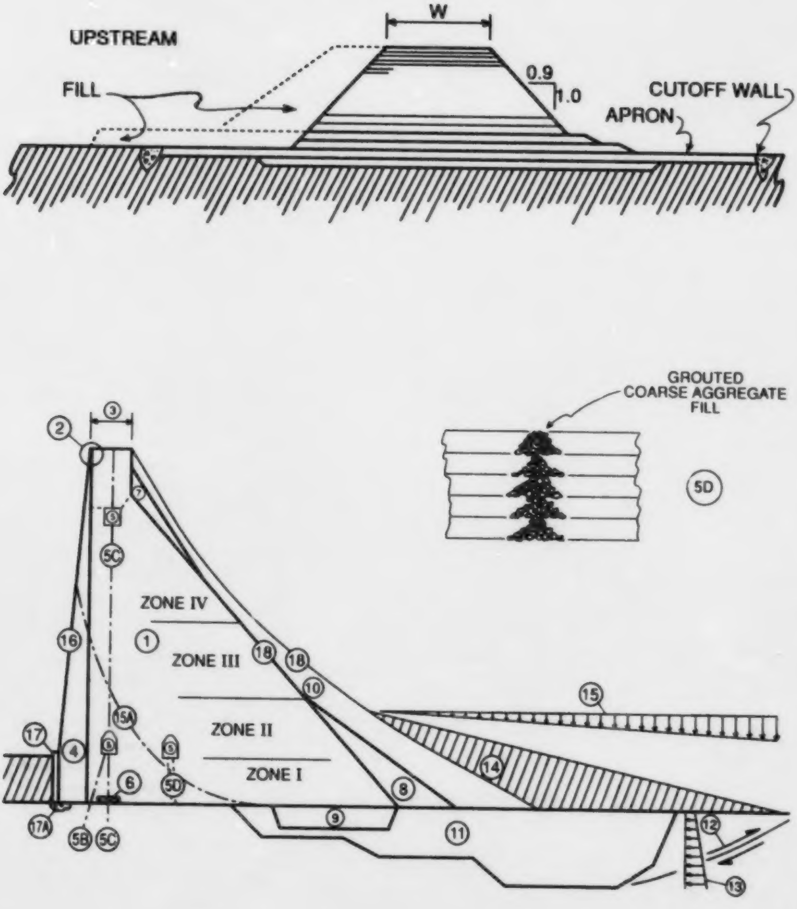


Figure 5. Saco Dam

Figure 6. Concepcion Dam


W
UPSTREAM
0.9
1.0
FILL
CUTOFF WALL
APRON
GROUTED
COARSE AGGREGATE
FILL
5D
3
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5C
ZONE IV
1
16
ZONE III
18
18
10
15
15A
ZONE II
17
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ZONE I
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5B
5C
13

Figure 7. Dam Section Options

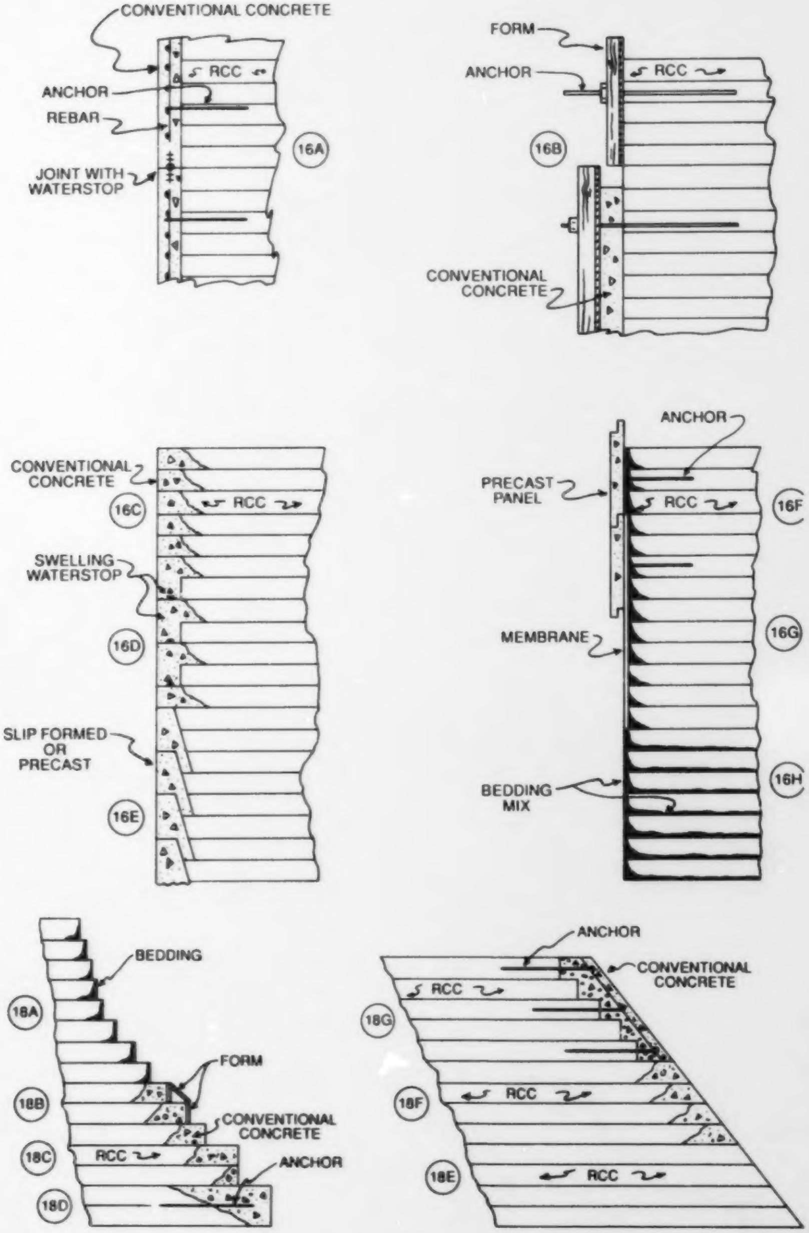
CONVENTIONAL CONCRETE
RCC
ANCHOR
REBAR
JOINT WITH WATERSTOP
16A
FORM
ANCHOR
RCC
16B
CONVENTIONAL CONCRETE
CONVENTIONAL CONCRETE
16C
RCC
SWELLING WATERSTOP
16D
SLIP FORMED OR PRECAST
16E
ANCHOR
PRECAST PANEL
RCC
16F
MEMBRANE
16G
BEDDING MIX
16H
BEDDING
18A
FORM
18B
CONVENTIONAL CONCRETE
18C
RCC
ANCHOR
18D
ANCHOR
CONVENTIONAL CONCRETE
RCC
18G
RCC
18F
18E
RCC

Figure 8. Facing Options

Rice Husk Ash--A Unique Supplementary Cementing Material

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Synopsis: The rice husk ash produced by controlled incineration is a highly active pozzolan, sometimes called a "superpozzolan." The material is very suitable for making high performance cement and concrete products. This paper contains a review of physical-chemical characteristics of rice husks, and effect of combustion conditions of rice husks on pozzolanic properties of the ash. Results from several investigations on the use of rice husk ash as a supplementary cementing material are presented. Finally, a review of recent studies on properties of cements and concrete containing rice husk ash is given.

Keywords: rice husks, rice husk ash, non-crystalline, surface area, pozzolanic activity, strength, permeability

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INTRODUCTION

Rice, wheat, and corn are major crops for cereal production. The non-cereal parts of these crops, such as stems, leaves, and husks are generally considered a waste, and their large volume presents a serious disposal problem. Since open-field burning can reduce the bulky agricultural matter to manageable amounts of ash, this has been the usual method of disposal. The ash is incorporated back into soil. Where the costs of collection, storage and transport of a crop residue are not high, some of the material ends up as a cheap fuel source or as animal fodder.

Due to the growing public concern with environmental pollution and an increasing interest in conservation of energy and resources, the traditional practices of waste disposal are no longer acceptable to the society today. Open-field burning of crop residues is now banned in many countries because fire and smoke cause lung and eye diseases in addition to possible damage to plant life. Interest in pollution-free disposal of waste materials has given impetus to research and development of methods for producing energy and useful industrial products from crop residues.[1]

For a variety of reasons rice husks (called rice hulls in the United States), which are a by-product of the rice paddy milling industry, have attracted more attention of cement researchers than other crop residues. First, in many rice-producing countries the size of the disposal problem is quite large and only cement and concrete industries can consume such large quantities of a by-product. Although rice husks make up about one-fifth (by weight) of the harvested and dried rice paddy, due to their very low bulk density (90-150 kg/m^3) heaps of the material can be seen awaiting disposal at the site of centralized milling operations. The current yearly world production of rice paddy is approximately 500 million tonnes. This means that about 100 million tonnes of rice husks are available for disposal every year.

Second, the economic disposal of rice husks has proved to be difficult due to their low value. This is because the material possesses rough and abrasive surface, has poor nutrition property as animal fodder, and is highly resistant to natural degradation. Also, it produces a high ash content on burning. On the average each tonne of rice husks, on complete combustion, produces 200 kg of ash. The data in Table 1 show that no other corp residue produces as much ash on burning. Third, the use of rice husk ash as a supplementary cementing material is of great interest to many developing countries where portland cement is in short supply but rice production is in abundance. World rice paddy and rice husk production data for important rice producing countries are shown in Table 2. It should be noted that every tonne of rice production corresponds to approximately 40 kg of ash. Cook[2] has estimated that about 1/3 of the available husks can be economically collected for incineration to ash, which may then be used as a supplementary cementing material.

This paper is a state-of-the-art report containing a review of physical-chemical characteristics of rice husk, effect of incineration conditions on pozzolanic properties of the ash, a summary of research findings from several countries on the use of rice husk ash as a supplementary cementing material, and recent work showing the uniqueness of properly burnt ash as a super-pozzolanic material.

RICE HUSK CHARACTERISTICS

The harsh woody covering or shell around the rice kernel consists of two interlocking halves. According to Houston,[3] each half consists of four structural layers: (1) the outer epidermis coated with a thick cuticle of highly silicified sinuous cells, among which the surface hair are found; (2) hyperderm fibers, also with thick and somewhat lignified and silicified walls; (3) spongy parenchyma cells; (4) inner epidermis of generally isodiametric cells.

The silica in rice husk is believed to exist in the opaline form (an amorphous, hydrous form of silica). Apparently, silica is taken up from soil and transported into the rice plant as monosilicic acid which becomes concentrated in husks and straw by evaporation, and finally polymerizes to form a cellulose-silica membrane. A scanning electron micrograph illustrating the typical cellular structure of rice husks is shown in Fig. 1. Also, from scanning and transmission electron micrographic studies it has been confirmed that silica is intimately dispersed throughout the cellular structure.

The average chemical composition of rice husks on anhydrous basis corresponds to 50% cellulose, 30% lignin, and 20% silica. The lignin and cellulose present in rice husks can be removed by controlled combustion, leaving behind silica ash in a microporous form, as discussed next.

CHARACTERISTICS OF RICE HUSK ASH

The partially burnt rice husk ash (RHA) contains carbon, and is therefore black in color. The fully burnt ash may be gray, purple, or white, depending on the impurities present and the burning conditions. As reported by Houston,[3] the chemical analyses of fully burnt RHA samples from several countries around the world show that the silica content ranges between 90 to 95%. The alkalies, K_2O and Na_2O, occur as principal impurities. The K_2O content of an RHA sample may range between 1 to 5%, depending on the type and amount of fertilizers used for rice growing. Smaller amounts (i.e. less than 1% each) of other impurities, such as CaO, MgO, and P_2O_5, have also been reported from some RHA samples.

Experimental studies at the University of California, which are described in detail below, showed for the first time that, when the temperature of burning rice husks is low and the duration of high-temperature exposure is kept relatively short, the silica in the ash retained its amorphous character. The product also showed high internal porosity indicated by 50 to 100 m^2/g surface area by nitrogen adsorption. This ash was found to be highly pozzolanic, and therefore an excellent supplementary cementing material. Also, particulate materials containing silica in a non-crystalline form are not considered a health hazard. With increase in the combustion temperature and time, the silica in RHA is transformed into crystalline forms, such as cristobalite and tridymite. The ash consisting mostly of crystalline silica shows much less pozzolanic activity, and is also considered hazardous to human health.

The bulk density of rice husks is 0.1g/cc (6.24 lb/ft^3) whereas the bulk density of rice husk ash is typically 0.2g/cc (12.5 lb/ft^3) which, for handling purposes, can be increased further by grinding.

REVIEW OF RESEARCH ON THE USE OF RHA AS A SUPPLEMENTARY CEMENTING MATERIAL

According to Cook,[2] before 1970s the use of RHA in the cement and concrete industry was limited to block making. Apparently, prior to 1970 no attempts were made to produce a pozzolanic ash by controlling the combustion conditions. Since the ash produced by uncontrolled combustion (both in field burning operations and in traditional industrial furnaces using rice husks as a fuel source) is generally crystalline and poorly pozzolanic, the products containing this type of ash and lime or portland cement require steam curing in order to develop adequate strength. It should be noted that in the concrete block industry, steam curing is a standard process for strength development in mixtures containing ordinary sand and lime or portland cement. In other words, high-temperature RHA is no better than ordinary sand as far as pozzolanic behavior is concerned.

In his comprehensive literature review on RHA, Cook[2] credits Mehta[4] for publishing in 1973 the first of several reports describing a careful study of the effect of pyro-processing parameters on the pozzolanic activity of rice husk ash. Based on Mehta's findings, Pitt[5] designed a fluidized-bed furnace for controlled burning of rice husks. In the Mehta-Pitt process, the heat from the combustion of rice husks was utilized to produce steam or electricity, and a close control of the time-temperature parameters in the burning operation made it possible to produce a highly pozzolanic ash. Blended cements made by mixing the RHA either with portland cement or lime showed excellent properties.[4] The idea that energy and a high quality supplementary cementing material can be produced by combustion of rice husks provided the impetus for research and development on this subject in many rice-growing countries particularly in South and Southeast Asia. Beginning with Mehta's work at the University of California, a review of research activities on the use of RHA as a supplementary cementing material in various parts of the world is presented next.

Research at the University of California, Berkeley

Based on research at the University of California at Berkeley, several published reports[4-9] show that a highly pozzolanic RHA can be produced by maintaining the husk combustion temperature at 500 to 700°C for relatively long periods (until most of the carbon is removed by oxidation). Alternatively, ash samples with similar characteristics were produced at temperatures as high as 700-800°C, provided that the high-temperature exposure was limited to less than one minute. The chemical analysis of the ash samples produced by a fluidized-bed type furnace showed 80-95% SiO_2, 1-2% K_2O, and 3-18% unburnt carbon. Significantly, the high-carbon RHA samples were found to be as pozzolanic as the low-carbon RHA samples. X-ray diffraction analysis confirmed that the ash produced under both these conditions was essentially amorphous (Fig. 2). Also, the product was highly cellular (Fig. 3), with 50-60 m^2/g surface area by nitrogen adsorption. The fibrous texture of the silica in ash results presumably from its deposition between cellulose fibrils in the original husk structure. Consequently, after the removal of cellulose the microvoids in the ash are also fibril-shaped (Fig. 3).

Since both the crystallinity of silica and microporosity of RHA are dependent on the combustion parameters, investigations were carried out on the effect of time and temperature of burning on rice husk ash characteristics. It was concluded that in order to obtain a product with high pozzolanic activity, not only should the silica be held in a noncrystalline state (disordered structure), but also in a highly microporous structure. The objective of combustion is, therefore, to remove by controlled oxidation the cellulose and lignin, while preserving the original cellular structure of the rice husk. The data in Table 3 show that above 800°C an increase in the combustion temperature, combustion time, and combustion environment tend to cause a sintering effect (coalescing of fine pores) which is indicated by a dramatic reduction in the specific surface. In regard to the combustion environment, it should be noted that a change in the rate of oxidation resulting from moderately oxidizing conditions (CO_2 environment) to highly oxidizing conditions (oxygen environment) was responsible for the steep drop in the microporosity and surface area (Table 3).

Based on Mehta's work[4] at the University of California at Berkeley, Pitt[5] designed a continuous process, fluidized-bed type furnace to produce energy and highly pozzolanic ash from the incineration of rice husks. In 1974, a 500 lb/h pilot plant was constructed in the Sacramento area in California, and subsequently a 7 1/2 t/h commercial plant was installed in Stuttgart, Arkansas. A flow diagram of rice husk burning plants based on the Mehta-Pitt process is shown in Fig. 4. It should be noted that a significant feature of the process is the utilization of fuel value resulting from the combustion of rice husks. About 2×10^6 J/kg of heat energy is released from rice husk combustion. Compressive strength characteristics of blended portland-rice husk ash cements containing 30, 50, or 70% ash from the Mehta-Pitt process are shown in Table 4. The cements not only showed high strength but also excellent resistance to acid attack (Fig. 5). Although the experimental units at Sacramento and Stuttgart are no longer in operation, the RHA cements produced by these units were successfully used for the construction of durable concrete floors in several fertilizer plants, refineries, meat packing facilities, and fruit and vegetable canning operations.

Mehta and Polivka[9] reported the use of rice husk ash as a highly active pozzolanic admixture for reducing expansion due to alkali-aggregate reactivity. In the ASTM C 441 standard test, whereas 25 percent by weight of an ordinary pozzolan (a calcined shale) was needed to reduce the expansion of the control mortar by 90%, only 10 percent of RHA was sufficient for obtaining the same degree of effectiveness. Also, compared to the mortar containing 25% calcined shale as a supplementary cementing material, the mortar containing 10% RHA did not show low strength at early ages. The results of this test proved that high-alkali cements can be successfully used in combination with alkali-reactive aggregates when a highly pozzolanic RHA, with amounts as low as 10% by weight of the total cementing material, is present as a partial cement replacement in the concrete mixture.

Research at the Asian Institute of Technology, Bangkok

Among others, Cook and his associates[10,11] reported the results of studies at the Asian Institute of Technology in Bangkok, Thailand, on the use of rice husk ash as a supplementary cementing material in portland cement concrete. In one of the studies, RHA was produced by burning rice husks at 450°C for 4 hours. The chemical analysis of the ash showed 93% SiO_2, 2.3% K_2O, and 3% ignition loss (residual carbon). The ash was pulverized to 32% residue on 75 µm sieve, however the surface area by nitrogen adsorption was not reported. Compressive strength data from cement-RHA mortars, with up to 50% cement replacement by RHA, are shown in Table 5. These strength values are significantly lower, when compared to Mehta's data in Table 4. This may in part be due to the higher water/cementitious ratios used by the authors.[10] In fact, the same authors reported that a concrete mixture, with 10% RHA cement and 0.49 water/cementitious ratio, gave 40.7 and 45.6 MPa compressive strength at 28 and 90-day, respectively. Similarly, a concrete mixture with 20% RHA cement and 0.53 water/cementitious ratio, gave 33.8 and 37.0 Mpa compressive strength at 28 and 90-day, respectively. These values are closer to Mehta's data and are significantly higher than those in Table 5.

Cook et. al[10] also investigated the creep, shrinkage, and swelling characteristics of concrete made with portland cement-RHA blends. The volume changes were found to be dependent on water/cementitious ratio, and were not significantly different than the control portland cement concrete when the cement replacement with RHA was limited to 30%.

Research in India

In 1974-76, research at the Indian Institute of Technology at Kanpur[12-13] in India was directed at the production of RHA-lime cements from the ash which is readily available in the country from either field burning of husks or from boiler-firing operations. As a result of uncontrolled combustion, the silica in this type of ash is in highly crystalline form, and therefore intensive grinding is necessary to produce even low-strength hydraulic cements. In fact, Mehta[14] had discussed the concept of lime-rice husk ash cements in an earlier publication (1971) on the production and properties of low-strength hydraulic binders produced by intensive grinding of mixtures containing lime and several pozzzolan types, such as calcined clays and shales, volcanic ash, and fly ash.

However, due to the high cost of grinding, very fine particle size (therefore high water requirement), and low strength, the cements of this type have not been produced commercially in the United States.

According to Chopra,[12] the cements developed by Kapur at the Indian Institute of Technology in Kanpur, made use of the ash which is produced by the combustion of rice husks as fuel for boilers in the parboiling method of dehusking rice paddy. The essential technology consisted of intergrinding in a ball mill for about 7 hours a mixture containing the boiler ash, freshly burnt quick lime, and slaked lime in the ratios 60:30:10, respectively. Prakash[13] reported the use of this type of cement for brick masonry work, plastering (stucco), and foundations. The author, however, acknowledged quality control problems attributable essentially to unsatisfactory and variable properties of raw materials used for cement making.

For specific application to developing countries, where rice husks are occasionally used as a cheap source of fuel in domestic cooking, Mehta carried out a preliminary study at the Indian Institute of Science in Bangalore. The objective of this study was to develop a small cooker, approx. 4 kg rice husks burning capacity, which can be used for cooking simple food for a family of four. Due to small heat capacity of the cooker, the temperature is automatically held low which keeps silica in the ash in a non-crystalline state. At the 1979 prices in South India, the cylindrical ring and cover assembly of the cooker (Fig. 7)[15] made of baked clay, cost approx. U.S. $0.50. Similarly, to produce a highly pozzolanic ash, Kapur[16] designed a tube-in-basket type rice husk burner. With incineration temperatures held automatically within 600 to 700°C range, the burner is able to produce a non-crystalline RHA with about 60 m^2/g surface area. Cristobalite was detected in the 800°C samples.

At the Cement Research Institute of India in New Delhi, Chopra[12] confirmed by X-ray diffraction analysis that the RHA obtained by combustion for two hours at 600°C contained silica in amorphous form, and it remained essentially amorphous for incineration temperatures up to 700°C. The authors confirmed that with increasing time and temperature of incinerations, silica tends to go into crypto-crystalline and then crystalline forms, thereby reducing the pozzolanic activity of the ash. Also, grindability studies showed that the non-crystalline ash was much easier to pulverize than the crystalline ash.

Even with highly pozzolanic RHA, the focus of research in India continued to be the production of masonry type, lime-RHA cements. However, Chopra[12] reported the compressive strength characteristic of portland-pozzolan cements containing 20% cement replacement. From his data, shown in Table 6, the author concluded that, at this replacement level, the strength properties of portland-RHA cements were equivalent to and even better than the control portland cement.

Research in Japan

Yamamoto and Lakho[17] reported the results of an investigation at the University of Tsukuba on production and use of active rice husk ash as a supplementary cementing material. RHA was prepared by burning rice husks under a free air supply condition at 400°C for 4 hours, cooled quickly, and ground for about 100 min. The physical and chemical characteristics of the ash are not described. However, the authors compared the pozzolanic behavior of this ash with the ash produced by Mehta,[7] and Cook et al.[10] The authors[17] made specimens of various portland cement-RHA combinations, with 20 to 70% cement replacement by RHA. The results of compressive strength tests at 3, 7, and 28-day are shown in Table 7.

Due to differences in the control portland cement, sand, and standard test methods (i.e., water/cementitious ratio, sand/cement ratio, curing conditions) a direct comparison of the three sets of data is not appropriate. However, according to the authors[17] strength changes in each mortar relative to its own control mortar (shown in parenthesis, Table 7) can be used as an index of pozzolanic activity of RHA. The authors concluded that their RHA was as active as that of Mehta's,[7] whereas the one used by Cook et. al[10] was of inferior quality. Similar to Mehta's results (Table 4), the authors found that even when 70% cement was replaced by RHA, the strengths were slightly higher than the control portland cement mortar.

Recent Research

Reviewed above are the important publications during the 1970s on the use of RHA as a supplementary cementing material. In this section, a summary of research on the subject during the 1980s is given. Since the number of published reports in the latter period is not large, it is not essential to discuss them separately.

Based on a M.Sc. thesis by Hana Yousef at the University of Technology, Baghdad, Iraq, Al-Khalaf and Yousif[18] described the effect of burning and grinding parameters on the properties of RHA in concrete. Rice husks were burnt in the 450°C to 850°C temperature range for periods ranging from 0.5 to 5 hours, and subsequently ground to 5000-21000 cm^2/g Blaine surface area. The authors concluded that burning at 500°C for 2 hours was optimum since most of the weight loss due to combustion of lignin and cellulose products had already occurred under these conditions (3.3% residual carbon). For standard consistency of mortar with a given RHA content, the water requirement decreased as the fineness of ash increased. Consequently, the minimum required value (75%) of pozzolanic activity index for a Class N pozzolan (ASTM C 618) was obtained only when the ash was pulverized to 11500 cm^2/g or higher Blaine surface area. The strength of portland cement-RHA mortar was similar to the strength of the control portland cement mortar only when the surface area of RHA was 17000 cm^2/g Blaine. With 21000 cm^2/g surface area, the pozzolanic activity index was about 110. At 30% cement replacement level, a blended cement containing a 11500 cm^2/g RHA (burnt at 500°C for 2 hours), gave mortar-cube compressive strengths at 7-, 28-, and 60-day that were similar to the control portland cement, in spite of the fact that the RHA cement mortar contained 25% more water content (0.71 vs 0.87 water/cementitious, respectively).

James and Rao[19] reported the results of a comprehensive study on the effect of burning conditions of rice husks on physical characteristics and chemical activity of the ash produced. Ash samples were prepared by heating the husks at 300, 400, 500, 600, 700, 800 and 900°C, for time periods ranging from 1-30h. The minimum thermal treatment to obtain carbon-free ash was 400°C for 12 hours. However, the product showed 3% loss of ignition due to the presence of hydroxyl water (Si-OH). Irrespective of the duration of heating, crystallization of silica did not take place below 700°C. The silica in the 800°C sample was slightly crystalline, however all the 900°C samples contained crystalline silica. The crystalline phase was identified as α-cristobalite, with a trace of tridymite. Although cristobalite is not a thermodynamically stable phase at this temperature, it can be stabilized in the presence of alkaline impurities, which are always present in rice husk ash.

The chemical activity of a sample of microporous RHA is governed not only by the surface area (which is mostly internal) but also by the crystallite size of silica. The authors[19] used X-ray line broadening technique to determine the crystallite size of silica. It was found that the crystallite size remained at ≈ 5nm (.005 μm) up to 700°C, but increased sharply in the 800-900°C temperature range. The surface area depended both on the temperature and the time of burning. With most hold-times, in the 1-18 hours range, the surface areas were lower at 300°C than at 400°C and 500°C, and decreased again with increasing temperatures. The increase of surface area between 300°C and 500°C was attributed by the authors to the liberation of silica from the husk, and the development of the pore structure. At 500 and 600°C burning temperatures, the surface area values remained quite high (100-150 m^2/g). At 700°C and 1-12h hold-time, the surface area was still above 50 m^2/g. However, a sharp drop was registered at 800°C and 900°C heat treatment. The decrease in the surface area with increasing temperature and hold-time was attributed to crystallite growth and pore structure collapse. The authors confirmed that the RHA samples with large surface area, small crystal size, and small particle size reacted faster with lime, whereas ashes with low surface area and containing silica in crystalline forms showed low reactivity.

Boatend and Skeete[20] burned rice husks from Guyana in a specially designed, small size (50-60 kg husk capacity), incinerator and investigated the ash for pozzolanic activity. With burning temperatures mostly in the 600 to 700°C range (the highest recorded in one case was 894°C), combustion for almost complete carbon removal took 7-9 hours (3-7% residual carbon content). The ash was ground in a ball mill to pass 150 μm sieve. RHA-portland cement blends were tested for compressive strength in mortar and concrete mixtures at a constant water/cementitious ratio of 0.6. In spite of the coarse size of the RHA, the concrete specimens with the RHA cement containing 1:1 RHA-portland cement blend produced 3-, 7-, and 28-day compressive strengths that were quite similar to the control portland cement. In mortar cubes, at 50% cement replacement level, the RHA gave approximately 30% higher strengths than the control.

Hwang and Wu[21] used rice husks from Taiwan to investigate the physical-chemical characteristics of the ash produced at different burning temperatures. According to the authors, the mean diameter of pores in RHA particles was highest for 600 and 700°C burning temperatures. Mercury penetration studies showed that there was a considerable drop in the total pore volume between 600°C and 700°C burning temperatures, therefore the ash made at 600°C was expected to show a higher pozzolanic activity.

Hwang and Wu[21] investigated the hydration characteristics and microstructure of cement pastes with varying water/cement and RHA/cement ratios. A cement paste made with 0.4 w/c ratio and containing 15% RHA, showed that just after 1-day of hydration most of the calcium hydroxide had probably reacted with RHA to form a dense spongy matrix. Fibrous CSH as a hydration product, formed within large pores of RHA. Compressive strength tests with ASTM C109 mortars (w/c 0.47) showed that, at the age of one day, the compressive strength of a cement containing 20% RHA was approximately 4.5 times when compared to the strength of the control portland cement. Hwang and Wu's data provide confirmation to an earlier report by Mehta[22] that among the known supplementary cementing materials, properly burnt RHA is probably the most reactive (Fig. 8). This provides a justification for the RILEM Technical Committee 73-SBC classification[23] which places RHA in the highly active pozzolan category. The highly pozzolanic character of RHA can be exploited to overcome the loss in early strength which is usually associated with the incorporation of normal pozzolans in concrete mixtures. According to data in Table 8,[24] a concrete mixture with 10% fly ash and 10% RHA, used as cement replacement, showed significantly less reduction in compressive strength at 3-, 7-, and 28-d than a corresponding mixture containing 20% cement replacement with fly ash alone.

It may be interesting here to speculate why Fig. 8 shows that RHA is the most reactive of all the supplementary cementing materials, in spite of the fact that condensed silica fume and many fly ashes and granulated blast-furnace slags also contain large amounts of non-crystalline reactive minerals. Unlike the low-temperature RHA (600-700°C), the above-mentioned materials are obtained from rapid cooling or condensation of droplets of high-temperature melts. The surface of products formed by condensation of a melt is not microporous, therefore an induction period is needed to activate the surface in the alkaline environment (provided by the hydration of portland cement). Apparently, in the case of RHA particles with highly microporous surface, a long induction period is not necessary for the surface reactions involving the formation of CSH and hydrous silica.

In regard to mechanisms responsible for improving the strength and other related properties of portland cement products containing a reactive RHA, a research study by the author[25] has shown that both physical and chemical processes are important. Introduction of very fine particles into a cement paste, even when the particles are not pozzolanic, leads to the segmentation of large capillary pores. Also, the availability of a large number of nucleation sites for precipitation of cement hydration products, not only accelerates the hydration reactions but also results in the formation of very small crystals of calcium hydroxide. In concrete both phenomena are known to contribute to high strength.[26] Further improvement in strength is caused by chemical activity, i.e., via the pozzolanic reaction between silica from RHA and calcium hydroxide liberated from the hydration of portland cement. In cement blends that are deficient in calcium hydroxide, some of the silica from RHA may hydrate to hydrous or opaline silica. The experimental confirmation of the pore-refinement phenomenon in cement pastes containing RHA was provided by Manmohan and Mehta (Fig. 9).[27]

The pore refinement process in the hydrated cement pastes, especially the paste in the immediate vicinity of the zone surrounding coarse aggregate particles in concrete, is a major contributor to the strength and impermeability of concrete. In this regard, the uniqueness of RHA as a superpozzolanic material is evident from the data in Tables 9 and 10, which are taken from the author's recent unpublished work.* A commercially produced RHA (91.3% SiO_2, 2.1% K_2O, 5% ignition loss, 75% residue on No. 200 mesh sieve, and 24.3 m^2/g surface area by nitrogen absorption), with silica in non-crystalline form, was used for this investigation. To obtain a homogeneous distribution of RHA in concrete, the sample was lightly ground to insure that approx. 90% particles were passing No. 200 mesh sieve (75 μm). Particle size analysis of the ground ash, using laser light-scattering, showed that 80% of the material was within the 10-75 μm particle size range.

The data in Table 9 show the mix proportions, and in Table 10 the properties of three high-strength, high-consistency, concrete mixtures with 10, 15, and 20% cement replacement by the RHA. Comparison between the RHA-containing mixtures and control mixtures showed the strength contribution effect of RHA even at early ages of 3 and 7-day. This effect is more pronounced at 28-d and 1-year. However, the corresponding effect on the impermeability of concrete is surprisingly high, and its magnitude was totally unexpected. For instance, the 28-d compressive strength of concrete containing 10 percent cement replacement with RHA was 18% higher than the control concrete, however the 28-d permeability of the former was almost one-third of the permeability value of the control (i.e. 1250 coulombs vs. 3500 coulombs). Note that resistance to chloride ion penetration, as measured by the 6-h AASHTO T-277 Standard Test, is accepted as an indirect index of the permeability of concrete. With 20 percent cement replacement by RHA, the 28-d strength increase was approx. 14% higher, however the corresponding permeability dropped to almost one-eighth (from 3000 to 390 coulombs). The 1-year data are even more impressive. By modifying the RHA characteristics slightly, the author has obtained a concrete with less than 50 coulombs value. No combination of admixtures or processing techniques in concrete technology are known to yield a product with such a low chloride permeability value. The potential usefulness of RHA, as a cement or concrete additive, for applications where the corrosion of reinforcing steel is a major concern, should be obvious from the results of this study.

CONCLUDING REMARKS

Worldwide, approximately 100 million tonnes of rice husks are available every year as a waste product from the milling of rice paddy. For environmental and economic reasons, their disposal by controlled combustion in industrial furnaces is an attractive idea, which can be used for the simultaneous recovery of potential energy and pozzolanic silica. About 2.0×10^6 J/kg of heat energy is produced from the rice husk combustion process, which leaves behind 18-20% of a highly siliceous ash. Depending on the efficiency of combustion, the silica content of the ash may be as high as 90-95%, with residual carbon being the main remaining ingredient. In addition to residual carbon, alkalies ranging from 1 to 3% usually form the other principal impurity.

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*Mehta, P. K., Highly durable cement products containing siliceous ashes (patents pending).

During the last twenty years several investigations have been undertaken on the effect of combustion parameters on pozzolanic characteristics of rice husk ash. A comprehensive review of published reports shows that, by limiting the time and temperature of combustion, it is possible to keep the silica in an amorphous and highly cellular form. Since crystalline silica is hazardous to human health, government agencies involved with environmental protection are increasingly demanding that the silica ash produced from incineration of crop residues be held in non-crystalline form. Therefore, non-crystalline silica ash from rice husks combustion, with 50-100 m^2/g surface area, is now available from many industrialized burning operations. Test reports from different research laboratories around the world have conclusively shown that this type of ash is highly pozzolanic.

To develop pozzolanic activity in the presence of lime or portland cement, the rice husk ash particles need not be very fine. This is because the source of high surface area in rice husk ash is in the microporous structure of individual particles. This is not the case with condensed silica fume which also happens to be a highly pozzolanic material. The surface area of silica fume, typically 20 m^2/g, comes mostly from the very fine particle size. The particles in silica fume are typically in the 0.01 to 1 μm range, and therefore are relatively difficult to handle and to disperse. On the other hand, rice husk ash particles, in the 10-75 μm range, exhibit satisfactory pozzolanic behavior. Also, it may be noted that the impurities normally present in rice husk ash do not cause any adverse effect on the properties of portland cement concrete.

As a supplementary cementing material, many unique properties of rice husk ash have been reported by several investigators. With fresh concrete mixtures pozzolanic additions, such as rice husk ash, have the ability to reduce bleeding and segregation, and thus cause significant improvement in workability. Except rice husk ash, no other pozzolanic material including silica fume has the ability to contribute to strength of portland cement concrete at the early ages of 1 and 3 days. This opens the door of using mixtures of fly ash or other normal pozzolans with rice husk ash, with the latter acting as a strength accelerator. Although up to 70 percent portland cement replacement has been reported without any adverse effect on strength, even 10 to 20% cement replacement shows a beneficial effect on strength and an extraordinary improvement in the chloride permeability characteristics of concrete. Also, blended portland cements containing 10% rice husk ash have been shown to be quite effective in combating expansion due to alkali-aggregate reaction.

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Table 1 Typical Ash Content of Major Crop Residues

Crop	Part of Plant	Ash (% by weight)
Corn	Leaf	12
Rice	Husk	20
Rice	Straw	14
Sorgum	Leaf	12
Sugarcane	Bagasse	15
Sunflower	Leaf and stem	11
Wheat	Leaf	10

Table 2 1990 World Production Rate for Rice Paddy and Rice Husk (million metric tons)

Country	Rice Paddy	Rice Husk
China	180	36
India	110	22
Indonesia	45	9
Bangladesh	27	5.4
Thailand	20	4
Vietnam	18	3.6
Burma	13	2.6
Japan	13	2.6
Brazil	9	1.8
Korea	9	1.8
Philippines	9	1.8
USA	7	1.4
Others	40	8
Total	500	100

Table 3 Effect of Combustion Conditions on the Crystal Structure and Surface Area of Rice Husk Ash (Ref. 8)

Combustion Temperature	Hold Time	Environment	Properties of Ash	
			Crystalline	Surface Area, m^2/g
500-600°C	1 min	moderately oxidizing	non-crystalline	122
500-600°C	30 min	moderately oxidizing	non-crystalline	97
500-600°C	2 hrs	moderately oxidizing	non-crystalline	76
700-800°C	15 min-1 hr	moderately oxidizing	non-crystalline	100
700-800°C	15 min-1 hr	highly oxidizing	partially crystalline	6-10
> 800°C	> 1 hr	highly oxidizing	crystalline	< 5

Table 4 Compressive Strength[*] of Portland-Rice Husk Ash Cements (Ref. 7)

Proportion of RHA in the Blended Cement	Compressive Strength, psi (MPa)			
	3 days	7 days	28 days	90 days
30%	4690 (33)	6690 (47)	8630 (60)	9390 (66)
50%	3840 (27)	5740 (40)	8460 (59)	8920 (62)
70%	3530 (25)	5210 (44)	6280 (44)	7370 (52)
Control portland cement (ASTM Type I)	3290 (23)	4780 (34)	6230 (43)	7010 (49)

[*] ASTM C109 mix proportions are used.
Cement/standard sand = 1:2.75; water/cement = 0.5

Table 5 Compressive Strength* of Cement-RHA Mortars (Ref. 10)

Proportion of RHA in the Blended Cement	Water/Cementitious	Compressive Strength, (MPa)		
		7 days	28 days	90 days
0	0.60	14.5	15.2	16.9
10	0.63	11.0	15.2	16.4
20	0.72	10.7	13.8	15.5
30	0.77	10.0	14.5	16.3
40	0.78	8.3	14.5	15.9
50	0.80	5.9	6.5	8.2

*Cement/sand = 1:2.75

Table 6. Physical Characteristics of Portland-Pozzolana Cement (Ref. 12)
(20 Percent, Cement Replacement by RHA)

Sample designation	Specific surface of cement, cm^2/g (Blaine)	W/C	Compressive strength, kg/cm^2 (as per IS:4031 - 1968)		
			3d	7d	28d
Control	3,780	0.42	252	313	448
LA-1	5,500	0.43	182	272	435
LA-2	7,000	0.45	260	387	528
LA-3	8,500	0.47	280	491	608
BA-1	6,000	0.41	197	268	440
BA-2	7,500	0.43	205	308	476
BA-3	9,000	0.43	203	353	530

Table 7 Comparison of Pozzolanic Activity of RHA Produced by Various Investigators

$\frac{\text{RHA}}{\text{Cement + RHA}}$ (%)	Compressive Strength of Mortar*, MPa							
	Ref. 17			Ref. 7			Ref. 10	
	3 Days	7 Days	28 Days	3 Days	7 Days	28 Days	7 Days	28 Days
0	13 (100)	19 (100)	28 (100)	23 (100)	33 (100)	43 (100)	14 (100)	15 (100)
20	16 (123)	27 (142)	38 (138)	29 (125)	47 (140)	61 (141)	11 (74)	14 (91)
25	17 (135)	28 (147)	40 (141)	-	-	-	-	-
30	-	-	-	33 (143)	47 (140)	60 (139)	10 (69)	14 (95)
50	15 (115)	23 (124)	36 (129)	27 (117)	40 (120)	59 (136)	6 (41)	8 (43)
70	14 (108)	20 (106)	28 (100)	25 (107)	37 (109)	44 (101)	-	-

*Figure in parenthesis shows relative strength of cement-RHA mortar to the strength of respective control mortar with no RHA, expressed in percent.

Table 8 Mix Proportions and Compressive Strengths of Concretes Containing Either Fly Ash or a Mixture of Fly Ash and Rice Husk Ash [Ref. 24]

	Mix No. 1	Mix No. 2	Mix No. 3
Cement	613 (362)	500 (297)	500 (297)
Fly ash	-	124 (74)	62 (37)
Rice husk ash	-	-	62 (37)
Coarse aggregate	1840 (1092)	1860 (1104)	1880 (1115)
Fine aggregate	1250 (742)	1230 (730)	1240 (724)
Water	323 (192)	305 (181)	324 (192)
Water/cementitious ratio	0.53	0.49	0.52
Compressive strength, psi (MPa)			
3-d	1720 (11.9)	1310 (9.0)	1510 (10.4)
7-d	2930 (20.2)	2390 (16.5)	2530 (17.4)
28-d	4850 (33.4)	4060 (28.0)	4320 (29.8)
56-d	5860 (40)	5390 (36)	5950 (41)

Table 9 Mix Proportions and Properties of Fresh Concrete Containing RHA*

Mix No.	Mix Proportions,* kg/m^3					Properties		
	C	RHA	C.A.	F.A.	W	W/C	Air, %	Slump, mm
1a	392	-	1062	786	128	0.33	1.0	200
b	356	36	1062	786	128	0.33	1.5	225
2a	410	-	1044	786	128	0.31	1.0	240
b	356	54	1044	786	128	0.31	1.5	175
3a	428	-	1026	786	128	0.30	1.5	225
b	356	72	1026	786	128	0.30	1.5	200

*All mixtures contained a constant amount of a superplasticizer in order to obtain high consistency.

Table 10 Properties of Hardened Concrete Mixtures of Table 9

Mix No.	Compressive Strength, MPa				Permeability, Coulombs[*]	
	3-d	7-d	28-d	1-yr	28-d	1-yr
1a	45	56	65	80	3500	2200
b	42	56	77	86	1260	420
2a	47	60	66	80	3260	2200
b	45	60	80	92	870	250
3a	47	62	70	81	3000	1800
b	46	65	80	92	390	190

[*] Coulombs passed in a 6h standard test (AASHTO T-277), based on FHWA Report No. RD-81/119, Aug. 1981.

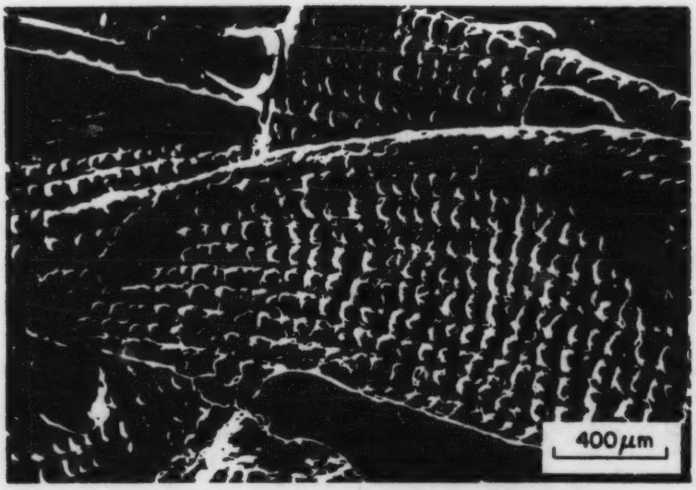

FIG. 1 SCANNING ELECTRON MICROGRAPH OF A RICE HUSK

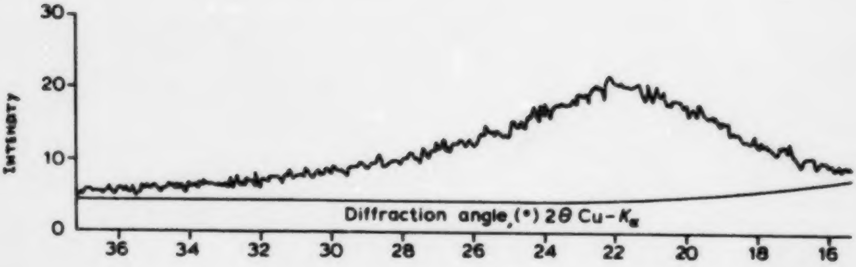

FIG. 2 X-RAY DIFFRACTION ANALYSIS OF NON-CRYSTALLINE RICE HUSK ASH

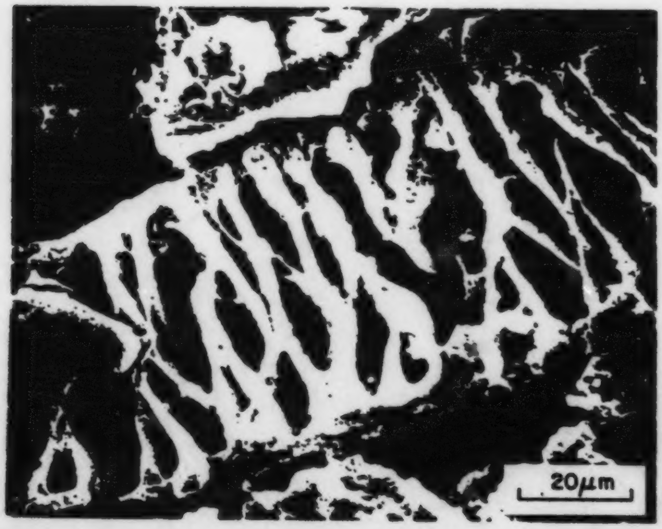

FIG. 3 SCANNING ELECTRON MICROGRPAH OF RICK HUSK ASH

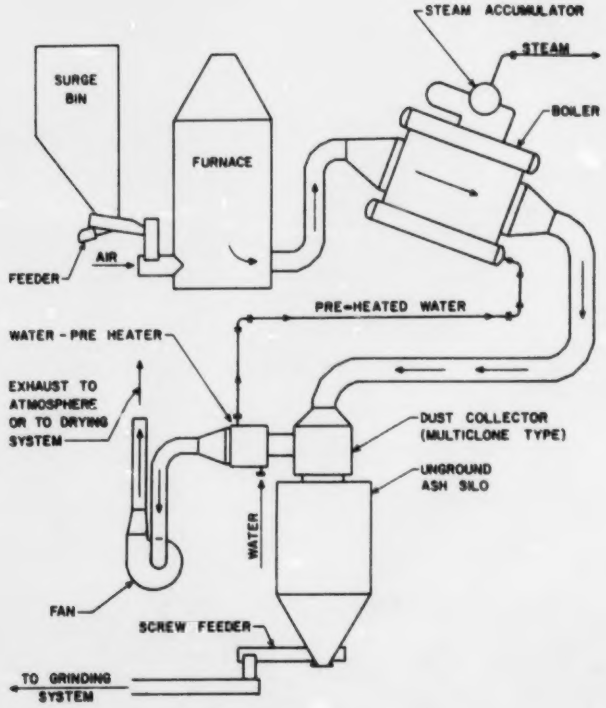
STEAM ACCUMULATOR
STEAM
BOILER
SURGE BIN
FURNACE
FEEDER
AIR
PRE-HEATED WATER
WATER - PRE HEATER
EXHAUST TO ATMOSPHERE OR TO DRYING SYSTEM
DUST COLLECTOR (MULTICLONE TYPE)
UNGROUND ASH SILO
WATER
FAN
SCREW FEEDER
TO GRINDING SYSTEM

FIG. 4 SCHEMATIC FLOW DIAGRAM OF PLANT FOR PRODUCING RH AND STEAM [REF. 1]

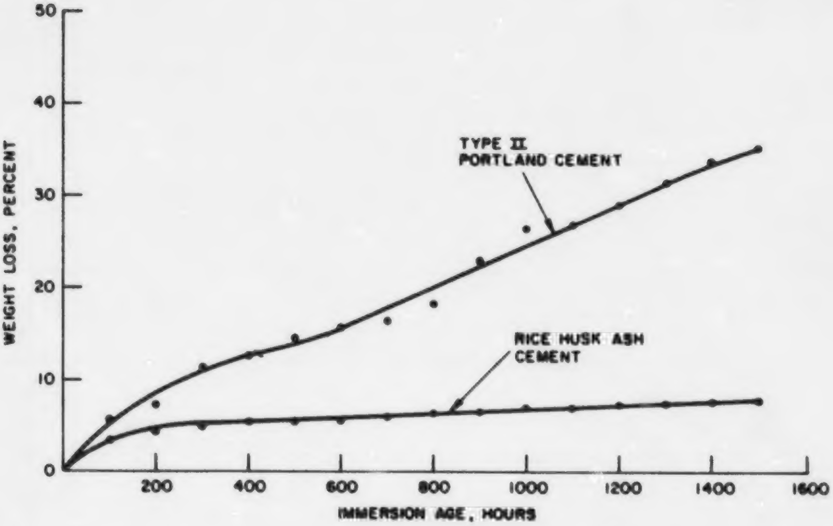

FIG. 5 RELATIVE WEIGHT LOSS OF PORTLAND CEMENT VS RHA CEMENT CONCRETE CYLINDERS (W/C = 0.4), CONTINUOUSLY IMMERSED IN A 5% HCl SOLUTION [REF. 6]

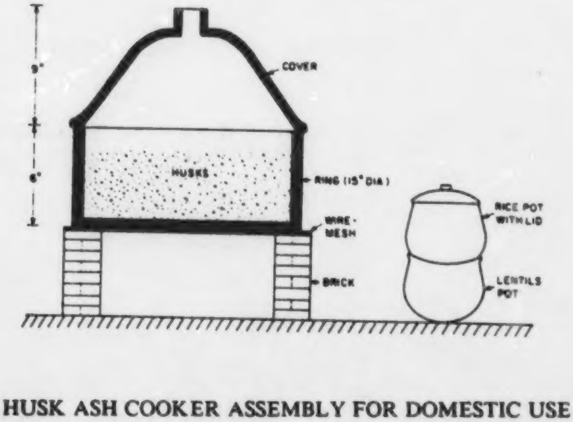
9"
6"
COVER
HUSKS
RING (15" DIA)
WIRE-MESH
BRICK
RICE POT WITH LID
LENTILS POT

FIG. 6 RICE HUSK ASH COOKER ASSEMBLY FOR DOMESTIC USE (REF. 15)

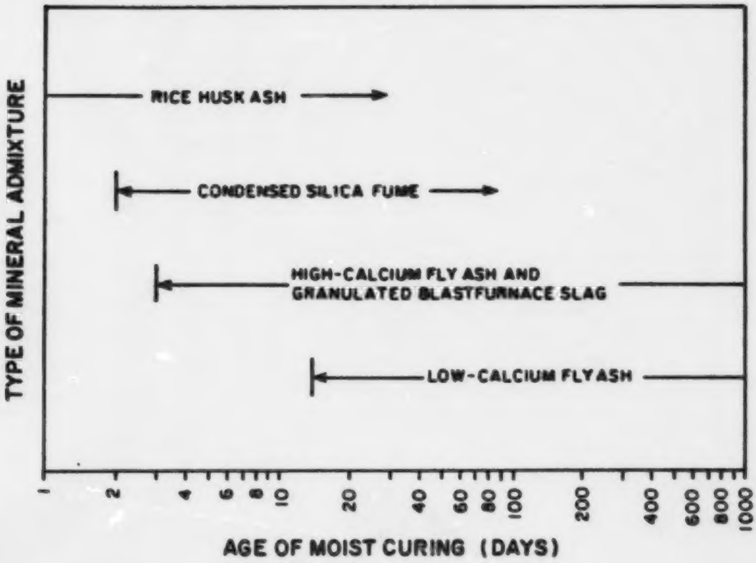

FIG. 7 RELATIVE AGE OF STRENGTH CONTRIBUTION BY VARIOUS SUPPLEMENTARY CEMENTING MATERIALS [REF. 22]

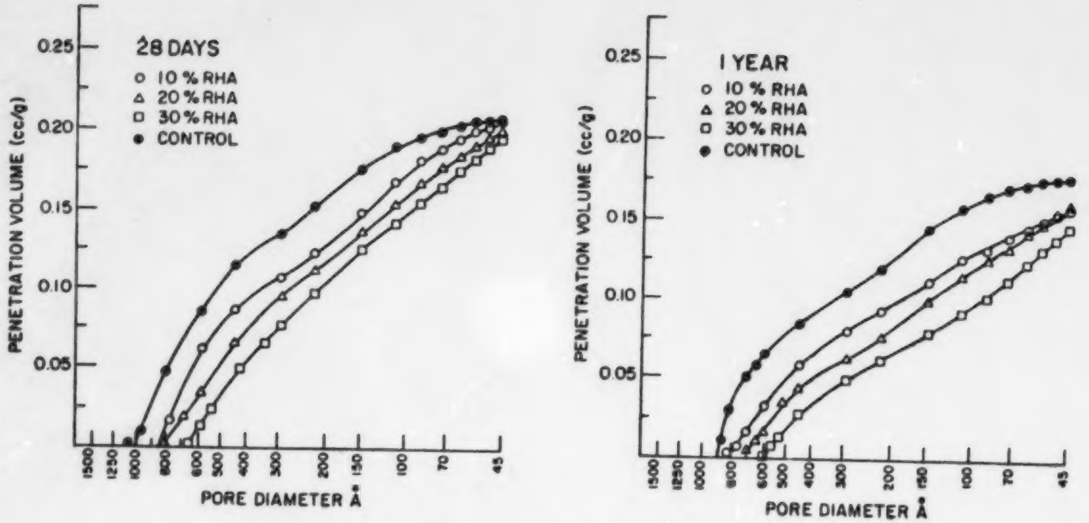

FIG. 8 PORE REFINEMENT IN HYDRATED CEMENT PASTES CONTAINING 10, 20, AND 30% RHA [REF. 27]

CANMET Investigations Dealing With High-Volume Fly Ash Concrete

by V.M. Malhotra

Synopsis: The Canada Centre for Mineral and Energy Technology (CANMET) has an on-going research program dealing with the use of supplementary cementing materials in concrete. As a part of this program, CANMET has developed concretes in which high volumes of low-calcium fly ash are incorporated and slumps in excess of 150 mm are obtained by the use of large dosages of superplasticizers. Typically, in high-volume fly ash concrete, cement content is kept at about 150 kg/m^3, the water-to-cementitious materials ratio is about 0.30, and fly ash content is between 56 and 60 per cent by weight of the total cementitious material.

This paper discusses the properties, applications and limitations of the high-volume fly ash concrete. The properties discussed include mixture proportions, bleeding, setting time, mechanical properties, heat of hydration, various aspects of durability, permeability and thermal properties. Also, mention is made of several structural applications of this type of concrete. It is concluded that, in general, high-volume fly ash concrete has excellent strength development and durability characteristics, and exhibits very low temperature rise. These properties make it ideal for use in large mat foundations, thick-structural elements and other similar applications.

Keywords: Concrete, Durability, Fly ash, Heat of hydration, Mechanism of hydration, Permeability, Strength, Superplasticizers, Thermal properties.

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INTRODUCTION

Low-calcium fly ash (ASTM Class F) has been incorporated into concrete as replacement for portland cement for the construction of mass gravity dams since 1935 (1). The primary aim in the use of fly ash was, and has been to reduce the heat of hydration. The low early-age strengths obtained by the use of fly ash were not critical as the compressive strength requirements were generally of the order of 20 MPa at 90 days or one year. In earlier years, the percentage of fly ash used was limited to about 30 per cent. In 1965, Mather (2) reported an investigation in which he studied the effects of 60% solid volume replacement of cement with fly ash. For water-to-(cement + fly ash) ratio (W/C+F) of 0.50 and 0.80, he reported three-day strengths of 4 and 1.5 MPa, respectively. In recent years, large volumes of fly ash have been incorporated into the construction of roller-compacted concretes but, once again, the low-early age strengths are not of concern (3).

In 1985, CANMET initiated studies on structural concrete incorporating high volumes (>50%) of low-calcium fly ash. The purpose of this research was to develop concrete with adequate early-age strength and workability, low temperature rise and high later-age strength. This was successfully achieved (4-19).

This paper summarizes the results of CANMET investigations dealing with the development of high-volume fly ash concrete, and discusses the applications and limitations.

MECHANISM OF DEVELOPMENT OF PHYSICAL AND MECHANICAL PROPERTIES AND HYDRATION REACTIONS

Feldman et al (4) and Berry et al (5) have performed investigations to explain the mechanisms of development of the physical and mechanical properties and hydration reactions in high-volume fly ash cement pastes and mortars. ASTM Type I cement and ASTM Class F fly ash were used in these investigations. Feldman et al performed investigations on fly ash/cement pastes at W/C+F of 0.30 and 0.35, and tested several hundred specimens (50-mm cubes, 0.9 x 31-mm discs, and 31.8 x 69.9-mm diameter cylinders) for compressive strength, modulus of elasticity, pore size distribution, total porosity and thermographic analyses. The following explanations were offered for the high-early strength of the high-volume fly ash/cement paste.

1a) The blended pastes at early ages have high effective water-to-cement ratio, allowing greater degree of hydration of the cement.

b) The fly ash commences reaction with the $Ca(OH)_2$ between 3 and 7 days.

c) Cement matrix and residual unreacted fly ash form a good mechanical bond.

d) The formation of a relatively homogeneous hydrate product with low $Ca(OH)_2$ content or the formation of low C/S ratio CSH produces a stronger body.

2) The volume of pores >0.9 μm of mixture containing high volumes of fly ash after 7 days of hydration is the same as the volume of pores of control mixture, also after 7 days of hydration. This suggests that durability properties may be adequate (Fig. 1,2).

3) Considerable amounts of $Ca(OH)_2$ and fly ash remain unreacted in all the fly ash mixtures after 91 days of hydration (Fig. 3,4).

Investigations performed by Berry et al. (5) were complementary to those performed by Feldman et al. In their studies, Berry et al. investigated the mechanisms of hydration in high-volume fly ash/portland cement binder materials at curing ages from 2 to 365 days. Two series of superplasticized mortars and pastes, one made with portland cement only, and the other with a mixture of 42% portland cement and 58% fly ash by mass were prepared at a water-to-cementitious material ratio of 0.30. Test specimens at the ages of 2, 5, 10 hours and 1,3,7,14,28,56,91, and 365-days were characterized physically and tested for compressive strength (mortars only) and pore volume. Also morphology by scanning electron microscopy, solid phase composition, degree of hydration by X-ray diffraction and thermal analysis, and pore fluid composition by high-pressure extraction were determined. Based upon the above research, the authors concluded as follows:

> "Overall, in the high-volume fly ash system, physical and chemical factors combine at all ages to densify and bind the paste. In the early stages, the physical effect of space filling and the chemical contribution of the formation of ettringite or related sulphoaluminate production are important factors in strength development. In the longer term, hydration reactions dominate as silico-aluminate binders are generated by reactions involving fly ash. The following observations indicate that chemical mechanisms are involved in late-age strength development in the portland cement-fly ash paste:
>
> (i) Levels of bound water substantially in excess of those consistent with the hydration of portland cement are present in the fly ash paste at ages beyond 28d. This is taken to indicate the formation of silanol groups in the fly ash glass components.
>
> (ii) The pore water solutions at curing ages beyond 1d have high concentrations of sodium, potassium and hydroxyl ions. These conditions are ideally suited to hydrolytic attack on fly ash glasses to produce silanol and related species.
>
> (iii) There is considerable evidence for a dynamic mass transfer of Na and K between solid and solution phases. This would be consistent with ion-exchange mechanisms occurring at silanol and siloxane sites in partly hydrolyzed fly ash glasses, or in gels precipitated from siloxane oligomers.
>
> It is suggested that a mechanism similar to the alkali-activation of alumino-silicates originally proposed by Purdon in 1940 is active in the fly ash pastes with the pore water acting as a vehicle for transfer of alkali ions from cement or fly ash into the fly ash glasses. Many glass types

are known to be hydrolyzed by strongly alkaline solutions with the formation of hydroxylated polymeric gels containing silanol, ionic Si-O^- ...Na+, or Al-OH groups. Similar reaction products have been found in previous work where fly ash was subjected to acidic hydrolysis.

Though the early attack on the fly ash glasses may derive from pore solutions of high Na^+ and K^+ content, it is probable that exchange with Ca^{2+} ions may subsequently proceed with the precipitation of C-S-H gel-like materials. These secondary processes may be responsible for the apparent decline in bound water content and the indicated gel-shrinkage responsible for the increase in pore volume at late ages.

While this investigation has indicated the importance of one or more chemical mechanisms in the action of fly ash in portland cement systems, it has also raised many questions about the role of alkali ions in the pozzolanic process. Further work is needed to elucidate the complex reaction sequences and processes that are undoubtedly involved."

MIXTURE PROPORTIONS

Like normal portland cement concrete, the mixture proportions of high-volume fly ash concrete will depend upon the strength level required. In general, the cement content varies from 100 to 180 kg/m^3 with W/C+F ranging from about 0.30 to 0.40. The water and fly ash contents are kept at about 115 and 220 kg/m^3, respectively, and slumps in excess of 150 mm are achieved with high dosages of superplasticizers. In studies at CANMET, the maximum size of coarse aggregate has been kept at 19 mm, but high-volume fly ash concretes have been made using 25 and 39 mm maximum size aggregate. The high percentage of fly ash in the concrete mixture does not pose any serious problems for the entrainment of 5 to 6 per cent air in concrete except that large dosages of the air-entraining admixtures are required. However, this has no adverse effect on the properties of fresh and hardened concrete (6). Nevertheless, exploratory investigations should be performed to ensure that air-entraining admixtures used are compatible with the type of cement and fly ash incorporated into the concrete mixture. Typical mixture proportions for the high-volume fly ash concrete are shown below:

	Batch Quantities
ASTM Type I Cement:	150 kg/m^3
ASTM Class F fly ash:	210 kg/m^3
Water:	115 kg/m^3
Coarse aggregate (19-mm max):	1275 kg/m^3
Fine aggregate:	620 kg/m^3
Air-entraining admixture:	720 mL/m^3
Superplasticizer:	4.0 L/m^3

The physical properties and chemical analysis of the various fly ashes used are given in Table 1.

PROPERTIES OF FRESH CONCRETE

Time of Setting

Data have been published on the initial and final times of set of high-volume fly ash concrete (11). In these investigations, the times of set were determined in accordance with ASTM C 403, at 23 ± 1.7°C and 50% relative humidity; the test results show that the initial times of set of 7h:30 are comparable to those of the control concrete, whereas the final times of set were retarded by about 3h when compared to that of the control. Other investigations have shown that undue delays in setting could occur for high-volume fly ash concrete made with certain fly ashes (14). These delays appear to be related to the problem of compatibility between cementitious materials and superplasticizer. This leads to very high dosages of superplasticizer which, in turn, create set retardation. The set-retardation, though not significant in general, may pose some scheduling problems in winter constructions; on the other hand, the delayed set can be of benefit during summer months. Some typical test data are shown in Table 2.

Bleeding

Bleeding tests performed on high-volume fly ash concrete have shown that this concrete does not bleed (Table 2). This is due to the very low water content (≈115 kg/m^3) used in this type of concrete (11). This would necessitate additional curing in hot weather to avoid plastic shrinkage cracks.

Entrainment of Air in Fresh Concrete

No difficulty has been encountered in the entrainment of air in high-volume fly ash concrete, though the dosage required is considerably more than that for portland cement concrete without fly ash (11,14). However, it should be pointed out that the fly ashes used in the high-volume fly ash concrete systems have relatively low carbon content. It may be difficult to entrain high percentages of air in the above system if the fly ash has carbon content >6%, but the judicious selection of an air-entraining admixture can overcome this problem (14).

Density of Fresh Concrete

Investigations at CANMET have shown that the density of high-volume fly ash concrete is of the order of 2400 kg/m^3, and is comparable with the density of portland cement concrete without fly ash. Considering that the specific gravity of fly ash is generally lower than that of portland cement (2.6 versus 3.1), the density of high-volume fly ash concrete is considered high, and this is due to the reduced water content due to the use of superplasticizers.

Dosage Requirement of Superplasticizer

Because of the very low water-to-cementitious material ratio (≈0.30) of the high-volume fly ash concrete, the use of superplasticizers becomes mandatory. The dosage required will depend upon the slump to be achieved; for flow concrete, the dosage is of the order of 1.5 per cent of the total cementitious material (≈5 kg/m^3). There is also a question of compatibility between superplasticizers and portland cements in the presence of fly ashes. Exploratory investigations should always be performed to ensure that there is no undue delay in the setting time of concrete when using the high-volume fly ash concrete system (11,14).

PROPERTIES OF HARDENED CONCRETE

Temperature Rise

Because of the very low cement content, the temperature rise in the high-volume fly ash concrete during the first few days after placement is very low. For example, the peak temperature measured at mid-depth of a high volume fly ash concrete block (1.5 x 1.5 x 1.5 m) at 2 days was 31.3°C when the ambient and placing temperatures were 24° and 12°C, respectively (12). Fig. 5 shows a comparison of computed and measured temperature distributions at mid-depth of the block. In another instance, in a concrete block measuring 9 x 7 x 3 m, the peak temperature of 37.5°C was reached after 7 days, when the placing and the ambient temperatures were 12°C and 24°C, respectively (12). Fig. 6 shows a comparison of the measured and completed temperature variations with time of the centre of the block. In another research project, the maximum temperature of 54°C was reached at the mid-height of a large high-volume fly ash concrete block, 2.5 x 4.0 x 5.0 metre in size, after 7 days of placement. The ambient and placing temperatures were 22 and 21°C, respectively (17). Fig. 7 shows a view of the blocks after casting. Thus the high-volume fly ash concrete is ideally suited for the construction of massive structures such as concrete dams; furthermore the placement of concrete can be done in one continuous lift rather than in 1.5 or 2.5 m lifts as in the current practice. This can result in substantial savings in construction costs.

Strength Properties

The high-volume fly ash concrete exhibits adequate strength development characteristics both at early and later ages. For example, the one-day compressive strength of high-volume fly ash concrete can vary from 5 to 9 MPa depending upon the type of cement. These strength values are more than adequate for formwork removal at normal temperatures, and are comparable to the strength developed by portland cement concrete with about 250 kg/m^3 of cement. The later-age compressive strength in some instances have been shown to reach about 60 MPa at one year. The 28-day flexural and splitting-tensile strengths of the high-volume fly ash concrete are about 15 and 10 per cent of the corresponding 28-day compressive strength, and these are, once again, comparable to the values for the normal portland cement concrete. Fig. 8 shows typical compressive strength development data for high-volume fly ash concrete.

The compressive strength development of air-cured high-volume fly ash concrete preceded by 7 days of initial moist curing as compared with the continuously moist-cured specimens follows the same trends as the control concrete i.e. the strength of the former concrete (air cured) is somewhat lower than the latter concrete (moist cured) (Fig. 9).

In-situ Strength Development of High-Volume Fly Ash Concrete

Investigations have been performed to determine in-situ strength development of high-volume fly ash concrete (17,18,19). In one investigation, six concrete mixtures of various compositions were investigated (18). The first five mixtures included a control mixture, mixtures with and without slag and or silica fume and mixtures incorporating a cementitious materials content of 485 kg/m^3 of concrete. The sixth mixture was a typical high-volume fly ash concrete incorporating a cement content of 150 kg/m^3 concrete and 200 kg/m^3 of fly ash. The concrete was obtained from a commercial ready-mixed concrete plant in the Toronto area. For each mixture, three types of structural elements simulating a thick-wall (0.5 x 2.5 x 1.5-m high), a thin-wall (0.25 x 3.1 x 1.5-m high) and a thick column (2.0 x 1.2 x 1.35-m high) were fabricated for testing under field curing conditions. Cores, 100 x 200-mm in size, were drilled at ages up to one year for determining the in-situ strengths of various concrete elements. In addition, a number of 150 x 300-mm cylinders were cast from each mixture for long-term strength testing.

The mixture proportioning and strength data for the test cylinders and the drilled cores are shown in Tables 3 and 4. Fig. 10 show the plots of compressive strength development of concrete incorporating portland cement and high volumes of ASTM Class F fly ash. The test data show excellent in-situ strength development of high-volume fly ash concrete. The in-situ strength as measured by drilled cores equal or exceed the control concrete and concrete incorporating slag and or silica fume (Table 4).

Bisaillon et al (17) and Langley et al (19) have also reported data on in-situ strength development of high-volume fly ash concrete.

Bisaillon et al (17) used both drilled cores and pulse velocity measurements to estimate in-situ strength of concrete blocks measuring 2.5 x 4.0 x 5.0 metre. Langley et al (19) determined the in-situ strength development by drilling and testing cores from large blocks measuring 3.1 x 3.1 x 3.1 metre in size. In each of the above investigations, the compressive strength of the drilled cores was comparable to or greater than the strength of the cores from the control concrete blocks.

Young's Modulus of Elasticity

The Young's modulus of elasticity of high-volume fly ash concrete made with limestone aggregate generally exceeds 30 GPa, and is thus somewhat higher than the modulus for comparable strengths of portland cement concrete. The higher modulus is probably due to large percentage of aggregates, and in addition, due to the unhydrated fly ash particles acting as a fine filler material in the concrete.

Creep Characteristics

The creep characteristics of high-volume fly ash concrete are shown in Fig. 11. The creep strain at one year ranges from 150 to 400 x 10^{-6} which is comparable to or lower than that of the portland cement concrete of comparable strength.

Drying Shrinkage

The drying shrinkage strains of high-volume fly ash concrete and normal portland cement concrete, determined in accordance with ASTM C 157, are comparable (11). Fig. 12 shows such data on drying shrinkage strains on fly ash and control concrete prisms which had been cured in lime-saturated water for either 7 or 91 days and then air dried at 23 ± 1.7° and 50 ± 4 per cent relative humidity for periods up to about 500 days.

Water Permeability

Water permeability tests using a CANMET uniaxial flow apparatus were performed on the fly ash concrete incorporating 150 kg/m^3 of ASTM Type I cement and 190 kg/m^3 of fly ash. Briefly, the test method consists of measuring the uniaxial water flow through concrete cylinders, 125 mm high with a diameter of 150 mm, under a pressure of 3.5 MPa. The test specimens were maintained in the permeability cells for more than 6 months, and it was found that no water had passed through the specimens during this period. Based upon data previously published by CANMET, this indicates that the concrete tested has extremely low permeability, although no numerical values can be assigned (20).

DURABILITY ASPECTS OF HIGH-VOLUME FLY ASH CONCRETE

Resistance to Repeated Cycles of Freezing and Thawing

A number of investigations have been performed to determine the resistance of high-volume fly ash concrete to repeated cycles of freezing and thawing (6-10). The test adopted was ASTM C 666, Procedure A, "Rapid Freezing and Thawing in Water". Briefly, this method consists of alternately lowering the temperature of specimens from 4.4 to -17.8°C and raising it from -17.8°C to 4.4°C in 4 h, resulting in about six cycles in a 24-h period. The durability of concrete is determined by measuring the resonant frequency of the test specimens before and after the cycling and calculating the durability factors. In CANMET investigations, additional parameters such as weight loss, length change and changes in pulse velocity caused by the freezing and thawing cycling of test specimens are also determined. Following the test, the reference and test prisms are broken in flexure.

The results of CANMET investigations show that properly air- entrained high-volume fly ash concrete performs more than satisfactorily in a freezing and thawing environment. The durability factors after 300 cycles of freezing and thawing range from 90 to 100. The bubble spacing factors $\bar{L}$ and the specific surface of hardened concrete are of the order of 0.20 mm and 20 mm^2/mm^3, respectively. Typical test data for one investigation are shown in Tables 5 to 8.

Resistance to the Penetration of Chloride Ions

Several investigations at CANMET have indicated that high-volume fly ash concrete has very high resistance to the penetration of chloride ions (6,7). The AASHTO T277-83 test method, "Rapid Determination of the Chloride Permeability of Concrete" is the most commonly accepted test in North America, and was therefore, adopted as the preferred technique. Briefly, the above method consists of monitoring the amount of electrical current passed through a 102-mm diameter by 51-mm thick concrete specimen when a potential difference of 60V dc is maintained across the specimen for a period of six hours. Chloride ions are forced to migrate out of a NaCl solution subjected to a negative charge through the concrete into a NaOH solution maintained at a positive potential.

The conditioning of the concrete disc specimens for the test procedure consists of one hour of air drying, three hours of vacuum (pressure <1 mm Hg), one hour of additional vacuum with specimens under de-aerated water, followed by 18 hours of soaking in water. The total charge passed, in coulombs, is used as an indicator of the resistance of the concrete to the passage of chloride ions.

The test results encompassing a number of investigations involving the use of several different fly ashes have indicated that the resistance of high-volume fly ash concrete to the penetration of chloride ions, as measured by the charge in coulombs, is very high. The values of the charge ranged from 150 to 973 coulombs for concretes tested at ages ranging from 28 to 91 days. It is generally agreed that for low permeability concretes, the value of the charge in coulombs passed through the specimens should not exceed 1000 and for very low permeability concretes this value should preferably be less than 600. In silica fume concrete incorporating 400 to 500 kg/m^3 of portland cement and 10 per cent silica fume, chloride-ion penetration tests yield charge in coulombs generally less than 800.

Resistance to Deicing Salt Scaling

CANMET has evaluated the scaling resistance of high-volume fly ash concrete to deicing salts (21). The concrete was evaluated using the ASTM test method C 672. Briefly, this test method consists of fabricating test specimens with a surface area of at least 0.046 m^2 and a depth of 75 mm. The test specimens are then moist-cured for 14 days, followed by 14 days of drying at a temperature of 23 ± 1.7°C and a relative humidity of 50 ± 5%. At the age of 28 days, the surface of the specimens is covered to a depth of 6 mm with a solution of calcium chloride. The test specimens are then placed in a freezing chamber for 16 to 18 h followed by thawing in laboratory air at 23 ± 1°C and a relative humidity of 50 ± 5% for 6 to 8 h. The procedure is repeated daily; visual examinations are made every five cycles and the test is continued up to 50 cycles.

The mixture proportions of the concrete investigated were somewhat similar to the typical mixture proportions described earlier. Instead of the 14 days of curing and 14 days of air drying periods as specified in the ASTM test, the curing and drying periods used were 28 and 42 days, respectively, and 3% sodium chloride solution was used instead of 4% calcium chloride solution. The curing and drying periods were extended to allow more maturity of fly ash concrete, and the sodium chloride solution was used because this is the most commonly used deicing agent in practice.

In addition to visual examination of the specimens, the weight of the scaled material was also determined. Both visual examination and weight loss determination indicated severe scaling of the surfaces of the test specimens with coarse aggregate visible over the entire surface. According to the ASTM scale of visual rating of 0 to 5[*], the test specimens were rated 5. Control plain portland cement concrete, made with the same water-to-cementitious materials ratio and the same total cementitious materials content has shown good scaling resistance with a visual rating of 1 which corresponds to slight scaling. Further research is indicated in this area.

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[*] The rating of 0 indicates no scaling and the rating of 5 indicates severe scaling (coarse aggregate visible over entire surface).

Carbonation

Limited data on carbonation tests performed on broken portions of cores drilled after five years from a large block of high volume fly ash concrete are shown in Table 9. The block was cast in December 1985 and moist cured for 28 days. Following that the block was left in a room with limited ventilation and at a temperature of about 23°C and a relative humidity of 40 to 50 per cent. The average carbonation depth was less than 10 mm after 5 years.

Control of Expansion Due to Alkali-Aggregate Reaction

The undesirable expansion of concrete due to reaction between the cement alkalies and certain types of silica in aggregates is a universal problem. Research at CANMET and elsewhere has shown that the above alkali-silica reactions in concrete can be controlled by incorporating good quality fly ash as a partial replacement for cement. The generally recommended levels of cement replacement by fly ash are between 25 and 40 per cent.

As described earlier, high-volume fly ash concrete incorporates about 56 per cent of fly ash as a percentage of total binder, together with large dosages of a superplasticizer. It was, therefore, considered prudent to perform investigations to ensure that the contribution of alkalies, both from the fly ash and the superplasticizers, will not adversely affect the ability of high-volume fly ash concrete to control or considerably reduce the above expansive reaction when reactive aggregates are used.

In order to determine the role of high-volume fly ash concrete in controlling expansion due to alkali-aggregate reaction, two concrete mixtures were made, one control and one incorporating ASTM Class F fly ash (13). The coarse aggregate used was 19-mm crushed limestone; this limestone contains a highly reactive silica phase and has a known history of expansive reaction in concrete.

A number of 75 x 75 x 305-mm prisms were cast and subjected to the following seven test regimes to determine the expansion due to the alkali-silica reaction.

Test regime 1: Continuous curing of the prisms in a moist-curing room maintained at 38°C.

Test regime 2,3:	Continuous curing of the prisms in 5% NaCL solution maintained at 38 and 80°C after an initial moist curing for 24 h.
Test regime 4,5:	Continuous curing of prisms in 1 normal NaOH solution maintained at 38 and 80°C after an initial moist curing for 24 h.
Test regime 6,7:	Continuous curing of prisms in 1 normal KOH solution maintained at 38 and 80°C after an initial moist curing for 24 h.

The prisms were exposed to the above regimes for a period of 275 days.

The test results given in Table 10 show that, regardless of the test procedure used, the test prisms cast from the high-volume fly ash concrete did not show any expansion in spite of the very reactive coarse aggregate used in the concrete. The above tests confirm the previously published data that fly ash can play an effective role in controlling the alkali-silica reactions in concrete.

THERMAL PROPERTIES OF HIGH-VOLUME FLY ASH CONCRETE

Thermal properties i.e. thermal conductivity, thermal diffusivity, and specific heat of high volume fly ash concrete at various moisture levels has been determined (22). The transient thermal probe method was used to determine the thermal conductivity (23) and the Shannon and Wells (24) technique was used to determine thermal diffusivity. Tables 11 and 12 show some of the test results together with the properties of fresh and hardened concrete.

APPLICATIONS/CASE HISTORIES

Concrete Block for Testing of Components for Communication Satellites, Ottawa

The first field application of the high-volume fly ash concrete was carried out in March 1987. This consisted of the casting of a concrete block, 9 x 7 x 3 m in size, at the Communication Research Centre in Ottawa. This block, cast indoors in permanent steel forms is being utilized in vibration testing of components for communication satellites, and was required to have as few microcracks as possible; also the compressive strength requirement was 40 MPa at 91 days together with the Young's modulus of elasticity value of not less than 30 GPa.

About 180 m^3 of high-volume fly ash concrete[*] was cast by pumping. The recommended placing temperature of the concrete and the ambient temperature were 7°C and 24°C, respectively. However, at the end of placement the temperature of the concrete had risen to 12°C due to delays in placing.

The peak temperature of 37.5°C was reached in the block after 7 days of casting. The values of compressive strength and the Young's modulus of elasticity, both tested on 150 x 300-mm cylinders at 91 days, were 46.1 MPa and 38.8 GPa, respectively. After 3 years the block is performing satisfactorily for the intended purposes.

Park Lane Hotel/Office Complex, Halifax

In 1988, a private developer in Halifax, Canada, completed the construction of an office and cinema complex in the city of Halifax. This development covers approximately 0.81 hectares and contains some 26,000 m^3 of concrete. The complex will provide 10,780 m^2 of office space contained in seven stories. A 600-car indoor parking garage also is included.

The lower columns, 900 mm diameter in size, and beams and floor slabs required 50-MPa concrete at an age of 120 days. The later-age strength requirements on this project were well suited for the use of concrete incorporating high volumes of fly ash. The following concrete mixture per m^3 of concrete was proportioned to satisfy the specified strength requirements.

Portland cement ASTM Type I	180 kg
Fly ash (ASTM Class F)	220 kg
Coarse aggregate (19-mm maximum)	1100 kg
Fine aggregate (natural sand)	800 kg
Water	110 L
Superplasticizer (naphthalene-based)	6 L

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[*] Mixture proportions: portland cement
ASTM Type II = 151 kg/m^3
fly ash (ASTM Class F) = 193 kg/m^3
C.A. = 1267 kg/m^3
F.A. = 668 kg/m^3
water = 125 kg/m^3
superplasticizer = 5.6 kg/m^3
A.E.A. = 680 mL/m^3

The exploratory mixture made in the laboratory yielded concrete with 74 MPa compressive strength at 120 days, which was significantly higher than the specified requirements; however, it was considered prudent to use a higher strength concrete mixture as the performance of the mixture in transit mixers had not been established.

When used in the field, the concrete mixture gave satisfactory strength results, and the envelope of all the tests to the age of 120 days is shown in Fig. 13. There were no unexpected problems with the concrete and the mixture proved to be economical for the 50 MPa compressive strength requirement at 120 days.

Purdys Wharf Development, Halifax

Phase II of the Purdys Wharf Development on the Halifax waterfront consists of a 22-story office tower founded on drilled caisson piles. There are 32 caissons of 1220 mm diameter and 30 caissons of 1066 mm diameter. The average length of the caisson is about 21 m. The piles are socketed into bedrock for a depth of 1.8 m. The piles were heavily reinforced, thus, a 14-mm nominal size of aggregate was proposed for use. A superplasticizer also was specified to increase the workability.

The contract specification for the pile foundation required a compressive strength of 45 MPa at 28 days. Concrete containing high volumes of low-calcium fly ash was selected for part of this project. The concrete mixture was basically similar to that of the Park Lane Development project discussed above, with the exception that the maximum size of coarse aggregate was 14 mm. Compressive strengths of the order of 32 and 51 MPa at 7 and 28 days, respectively, were obtained for this concrete. In situ load tests were also performed to determine the bond strength between concrete and rock. Bond strengths developed were shown to be quite satisfactory, with values exceeding 3 MPa, which was about 2.5 times the design criteria value of 1.2 MPa.

Polypropylene Fibre-Reinforced High-Volume Fly Ash Shotcrete

Polypropylene fibre-reinforced high-volume fly ash shotcrete[*] has been used to stabilize rock outcrop slopes on a highway in Nova Scotia, Canada. In this type of application, the total cementitious material content is kept at about 400 kg/m^3, 60 per cent of which is fly ash. Polypropylene fibres of 25 mm in length are incorporated into the mixture at an addition of 4 to 5 kg/m^3 together with high dosages of superplasticizer. The entrained air content is maintained at about 6 per cent. No long-term performance data are available on this application. However, similar shotcrete has been used on an experimental basis to cover several hundred square meters of horizontal pyritic shale rock fill at the Halifax International Airport, Halifax. The site is subjected to about 100 cycles of freezing and thawing per year. The experimental sections show satisfactory performance one winter after placement.

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[*] Personal communication from Wilbert S. Langley, W.S. Langley and Associates, Dartmouth, N.S., November 1990.

LIMITATIONS

The major limitations in the utilization of high-volume fly ash concrete is the availability of good quality fly ash within economic haulage of the construction site and adequate silo capacity. The other limitations include the compatibility between fly ash/cement and superplasticizer. This needs to be investigated as each source of fly ash is unique. In some instances the initial and final setting of the high-volume fly ash concrete may be retarded by several hours. This problem can likely be resolved by changing the type and brand of the superplasticizers used.

The performance of high-volume fly ash concrete subjected to de-icing salts is relatively poor and further research is needed before the use of this type of concrete can be recommended for this application involving this kind of exposure.

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Table 1 - Physical Properties and Chemical Analysis of Various Fly Ashes Used in High-Volume Fly Ash Concrete

	Fly Ash A	Fly Ash B	Fly Ash C
Physical Properties			
Fineness - passing 45 µm, %	78.8	82.7	80.6
- Blaine, m^2/kg	--	289	326
Specific Gravity	2.38	2.53	2.05
Chemical Analysis			
Insoluble residue	--	--	--
Silicon dioxide (SiO_2)	45.1	47.1	55.6
Aluminum oxide (Al_2O_3)	22.2	23.0	23.1
Ferric oxide (Fe_2O_3)	15.7	20.4	3.48
Calcium oxide (CaO), total	3.77	1.21	12.3
Magnesium oxide (MgO)	0.91	1.17	1.21
Sulphur trioxide (SO_3)	1.40	0.67	0.30
Sodium oxide (Na_2O)	0.58	0.54	1.67
Potassium oxide (K_2O)	1.52	3.16	0.50
Loss on ignition	0.32	2.88	0.29

Table 2 - Properties of Fresh Concrete+

Mixture Series	Batch No.	Temperature, °C	Slump, mm	Unit Weight, kg/m^3	Air Content, %	Setting Time h:min.		Bleeding cm^3/cm^2
						Initial	Final	
A(Control)	1	22	90	2375	5.9	7:20	8:40	*
	2	22	125	2410	4.9			
	3	23	90	2400	5.6			
	4	23	90	2410	5.0			
B(Fly Ash L)	1	24	175	2430	4.8	7:30	11:30	*
	2	21	200	2410	5.0			
	3	21	200	2420	4.7			
	4	21	200	2395	5.2			
	5	22	225	2375	5.2			
	6	23	200	2415	4.0			
	7	24	200	2400	4.6			
	8	25	175	2365	5.6			
	9	22	175	2410	3.8			
C(Fly Ash S)	1	25	175	2340	4.9	8:40	11:40	*
	2	25	150	2350	4.9			
	3	26	200	2345	4.6			
	4	24	190	2365	4.4			
	5	25	130	2420	3.7			
	6	23	200	2330	4.9			
	7	23	150	2340	4.5			

+From reference 11.

*No water accumulated on the surface when tested according to ASTM C 232

Table 3 - Mixture Proportions of Concrete Mixtures and Properties of Fresh Concrete (Non-air entrained)+

Mixture No.	Water/ Cementitious Materials	Quantities, kg/m^3									Properties	
		Water	Cement	Slag	Silica Fume	Fly Ash	C.A. (S.S.D.)	F.A. (S.S.D.)	S.P. (L/m^3)	S.R.A. (L/m^3)	Slump, mm	Unit Weight, kg/m^3
1 (CSSi)	0.29	143	315	135	35	-	1142	744	7.5	0.45	250	2400
2 (CS)	0.28	133	317	167	-	-	1145	749	7.0	0.46	110	2410
3 (C8Si)	0.27	130	449	-	39	-	1149	758	11.0	0.84	225	2420
4 (C12Si)	0.27	132	427	-	59	-	1139	754	14.9	0.90	200	2400
5 (CF)	0.29	102	150	-	-	200	1222	811	7.1	0.66	210	2440
6 (C)	0.27	130	485	-	-	-	1143	762	3.4	0.94	150	2464

+From reference 18.

NOTE: C.A. - Coarse aggregate, F.A. = Fine aggregate, S.P. - Superplasticizer,
S.R.A. - Set retarding admixture
S.S.D. - Saturated, surface dry

Table 4 - Summary of Compressive Strength Test Results+

	Compressive* Strength of 150 x 300-mm Cylinders, MPa						
	Age, days	1-CSSi	2-CS	3-C8Si	4-C12Si	5-CF	6-C
Moist Cured	1	36.7	34.5	38.4	43.5	8.3	44.4
	3	55.8	45.1	56.5	63.7	21.9	52.5
	7	69.6	57.9	66.9	77.7	33.7	58.5
	28	79.9	71.6	81.6	90.6	54.7	70.1
	91	81.1	75.7	84.3	92.2	69.5	85.0
	182	85.9	83.1	87.5	94.3	80.7	94.6
	365	88.6	82.7	91.5	103.7	97.5	100.8
Air Cured	1	39.6	34.8	47.8	53.4	**	45.9
	3	54.7	46.3	59.0	68.7	19.9	53.0
	7	62.1	53.0	68.5	73.5	28.3	59.5
	28	79.5	69.5	76.8	85.0	43.4	64.7
	91	82.1	62.9	80.6	91.7	57.6	79.4
	182	83.4	71.1	84.6	89.2	71.9	83.1
	365	85.7	74.6	85.2	98.1	75.4	88.0
	Compressive Strength of 100 x 200-mm Cores, MPa						
Thick-Wall Element A	1	51.0	43.4	57.5	61.6	**	38.7
(0.5 m x 2.5 m x 1.5 m)	3	62.4	51.1	66.2	-	24.3	50.8
	7	60.9	58.5	62.8	72.5	29.9	49.0
	28	69.2	64.8	70.6	74.8	52.1	57.2
	91	74.2	68.9	69.8	72.8	87.5	76.2
	182	73.3	70.2	76.4	81.0	92.1	80.2
	365	79.7	76.1	80.2	87.5	98.2	86.5
Thin-Wall Element B	1	48.5	46.0	57.0	67.2	N/A	47.4
(0.25 m x 3.1 m x 1.5 m)	3	60.7	53.2	65.7	-		52.0
	7	63.8	57.8	65.8	66.5		57.5
	28	73.1	68.6	72.3	75.9		64.3
	91	83.3	70.8	76.3	71.5		85.7
	182	80.7	70.3	80.2	84.8		86.7
	365	81.7	84.8	82.9	88.1		98.2
Thick-Column Element C	1	41.7	45.6	58.4	69.4	7.8	41.7[a]
(2.0 m x 1.2 m x 1.35 m)	3	64.3	55.0	63.5	-	27.1	47.5
	7	63.2	57.2	63.4	71.8	34.0	51.4
	28	69.4	63.4	66.8	75.8	49.9	59.9
	91	70.8	63.3	69.1	72.9	82.5	75.6
	182	69.6	69.4	71.4	78.7	87.0	74.3
	365	78.5	76.2	79.2	80.8	95.6	88.2

+From reference 18.

*Each strength value is the average of 3 test specimens

**Concrete not set at time of test

[a] Tested at 2 days

Table 5 - Concrete Mixture Proportions - Freezing and Thawing Investigation+

Mixture No.	W/(C+F)	Batch Quantities, kg/m³							
		Water	Cement	As Received Fly Ash	Beneficiated Fly Ash	Fine* Aggregate	Coarse* Aggregate	A.E.A., mL/m³	SP, kg/m³
1	0.33	119	154	212	-	622	1276	721	4.26
3	0.33	119	154	-	212	622	1276	792	4.26

+From reference 8.

*SSD condition

NOTE: A.E.A. = Air-Entraining Admixture; SP = Superplasticizer

Table 6 - Properties of Fresh and Hardened Concrete* - Freezing and Thawing Investigation+

Mixture Series	Properties of Fresh Concrete				Properties of Hardened Concrete	
	Temperature, °C	Slump, mm	Unit Weight, kg/m³	Air Content, %	Compressive Strength of 152 x 305-mm Cylinders at 28 Days, MPa	Flexural Strength of 76 x 102 x 406-mm Prisms at 28 Days
1	21	205	2400	4.3	31.6	4.2
3	23	235	2425	4.3	30.5	5.0

+From reference 8.

Table 7 - Air Void Parameters of Hardened Concrete* - Freezing and Thawing Investigation+

Mixture No.	Air Content of Fresh Concrete, %	Air Voids Parameters of Hardened Concrete			
		Voids in Concrete, %	Voids per mm	Specific Surface, mm^{-1}	Spacing Factor, mm
1	4.3	4.8	0.195	16.8	0.215
3	4.3	4.3	0.201	18.7	0.253

+From reference 8.

*ASTM recommended practice for microscopical determination of air-void content and parameters of the air-void system in hardened concrete (C 457)

Table 8 - Summary of Test Results on Concrete Prisms After 300 Cycles of Freezing and Thawing+

Mixture No.	W/(C+F)	Air Content, %	76 x 102 x 390-mm Test Prisms*													
			Weight, kg			Length, mm			Pulse Velocity, m/s			Resonant Frequency, Hz			Relative Dynamic Modulus of Elasticity, %	Durability Factor
			W_0	W_{300}	Change, %	L_0	L_{300}	Change, %	V_0	V_{300}	Change, %	N_0	N_{300}	Change, %		
1	0.33	4.4	7.443	7.333	-2.15	361.03	361.15	+0.033	4914	4627	-5.83	5500	5360	-2.60	95	95
3	0.33	4.4	7.453	7.380	-1.68	360.65	360.71	+0.017	4796	4618	-3.71	5410	5320	-1.64	97	97

+From reference 8.

NOTE: W_0, L_0, V_0 and N_0 represent values of weight, length, pulse velocity, and resonant frequency at the commencement of test; W_{300}, L_{300}, V_{300} and N_{300} represent test values at the end of 300 cycles of freezing and thawing

*Each result represents the average of two tests

Table 9 - Carbonation of High-Volume Fly Ash Concrete+

Type of Air-Entrained Concrete Specimen	Curing History	Test Age (Months)	Average Carbonation Depth (mm)
76 x 102 x 406-mm prisms[*]	Moist-cured for 7 days followed by air-cure at 23 ± 1.7°C and 50 ± 5% R.H.	12	3
102 x 203-mm drilled cores[**] from the 1.6 x 1.6 x 1.6-m block	Moist-cured for 28 days and afterwards left indoors at 23 ± 1.7°C and 50 ± 5% R.H.	31	4
		41	4
		50	7
		57	7

+From reference 12.

[*] Mixture Proportions: Water = 115 kg/m^3, ASTM Type I cement = 157 kg/m^3, ASTM Class F fly ash = 216 kg/m^3, coarse aggregate = 1284 kg/m^3, fine aggregate = 627 kg/m^3, superplasticizer = 5.6 kg/m^3, air-entraining admixture = 827 mL/m^3, compressive strength of 152 x 305-mm cylinders at 28 days = 36.4 MPa.

[**] Mixture Proportions: Water = 95 kg/m^3, ASTM Type II cement = 147 kg/m^3, ASTM Class F fly ash = 187 kg/m^3, coarse aggregate (dry) = 1215 kg/m^3, fine aggregate (dry) = 622 kg/m^3, superplasticizer = 7.0 kg/m^3, air-entraining admixture = 660 mL/m^3, compressive strength of 152 x 305-mm cylinders at 28 days = 41.6 MPa.

Table 10 - Changes in Length of Concrete Prisms Under Different Storage Conditions - Alkali-Aggregate Reaction Investigation+

Storage	Type of Mixture	% Expansion at Different Ages							
		7d	14d	28d	56d	84d	112d	168d	275d
1. Plain Water Temp. 38°C	Control*	0.005	0.008	0.003	0.028	0.056	0.063	0.078	0.087
	Fly ash**	-0.006	-0.005	-0.006	-0.001	-0.004	-0.004	-0.007	-0.004
2. 5% NaCl Temp. 38°C	Control	0.003	0.001	0.001	0.014	0.032	0.044	0.064	0.077
	Fly ash	0	-0.006	-0.005	-0.006	-0.001	-0.009	-0.006	-0.004
3. 5% NaCl Temp. 80°C	Control	0	0.018	0.062	0.064	0.083	0.086	0.104	0.130
	Fly ash	-0.011	-0.007	-0.005	-0.005	-0.004	-0.004	0.015	0.025
4. 1 Normal NaOH Temp. 38°C	Control	0	0	0	0.020	0.045	0.061	0.088	0.124
	Fly ash	-0.007	-0.010	-0.012	-0.010	-0.013	-0.010	-0.007	-0.005
5. 1 Normal NaOH Temp. 80°C	Control	0.020	0.059	0.090	0.108	0.158	0.196	0.270	0.347
	Fly ash	-0.011	-0.01	-0.010	0	0	+0.009	0.020	0.038
6. 1 Normal KOH Temp. 38°C	Control	0	0	-0.005	0.020	0.040	0.050	0.071	0.077
	Fly ash	-0.010	-0.010	-0.011	-0.003	-0.007	-0.008	-0.004	-0.003
7. 1 Normal KOH Temp. 80°C	Control	0.020	0.050	0.070	0.090	0.149	0.201	0.266	0.346
	Fly ash	-0.013	-0.016	-0.005	-0.005	-0.005	-0.003	0.014	0.016

+From reference 13.

*W/C = 0.32, water content = 115 kg/m^3, cement content = 365 kg/m^3, alkali content of cement = 4.12 kg/m^3, Superplasticizer = 9.3 kg/m^3, compressive strength = 47.8 MPa at 28 days, slump = 105 mm

**W/C+F = 0.31, water content = 114 kg/m^3, cement content = 155 kg/m^3 alkali content of cement = 1.75 kg/m^3, Superplasticizer = 6.2 kg/m^3, compressive strength = 36.4, slump = 145 mm

Table 11 - Properties of Fresh and Hardened Concrete: Investigation for Thermal Properties+

Mixture Number	Fresh Concrete				Hardened Concrete		
	Temperature, °C	Slump, mm	Unit Weight, kg/m^3	Air Content, %	Unit Weight At One Day, kg/m^3	Compressive Strength, MPa 102 x 203-mm Cylinders	
						7-day	28-day
1	23	75	2415	3.4	2465	--	36.7
2	25	200	2415	4.2	2460	22.0	34.8

+From reference 22.

Table 12 - Thermal Properties of Concrete+

Mixture Number	Moisture Content, %												
	0			1		2		3		4		5	
	TC*	TD**	SH***	TC	TD	TC	TD	TC	TD	TC	TD	TC	TD
1	1.56	89	0.171	1.75	95	1.96	101	2.08	105	2.13	98	--	--
2	1.55	89	0.169	1.72	94	1.90	98	2.04	100	2.13	99	2.17	97

+From reference 22.

*Thermal conductivity in W/°C-m
**Thermal diffusivity in cm^2/sx10^{-4}
***Specific heat in Cal/g.°C
Moisture content is given as a percent of dry weight

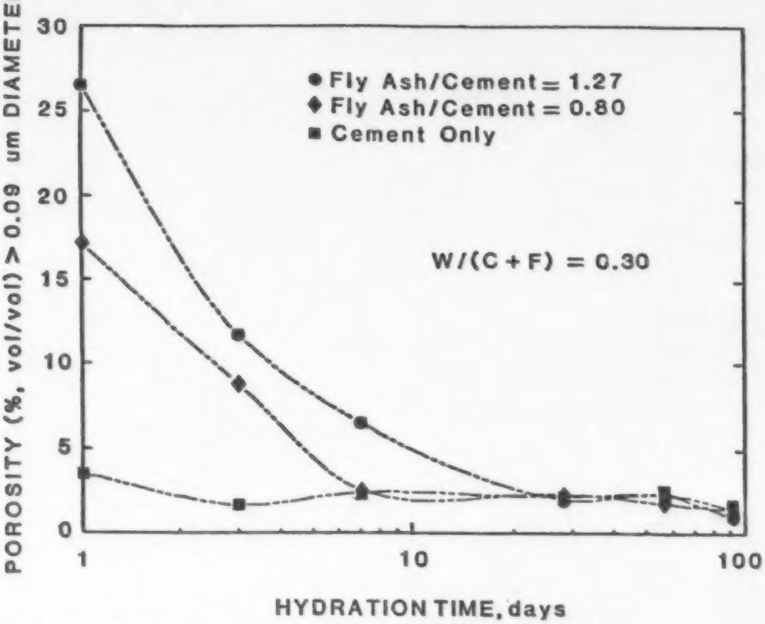

Fig. 1 - Volume of pores (>0.09 μm dia) versus hydration times for pastes with water-to-cementitious material ratio of 0.30. From reference 4.

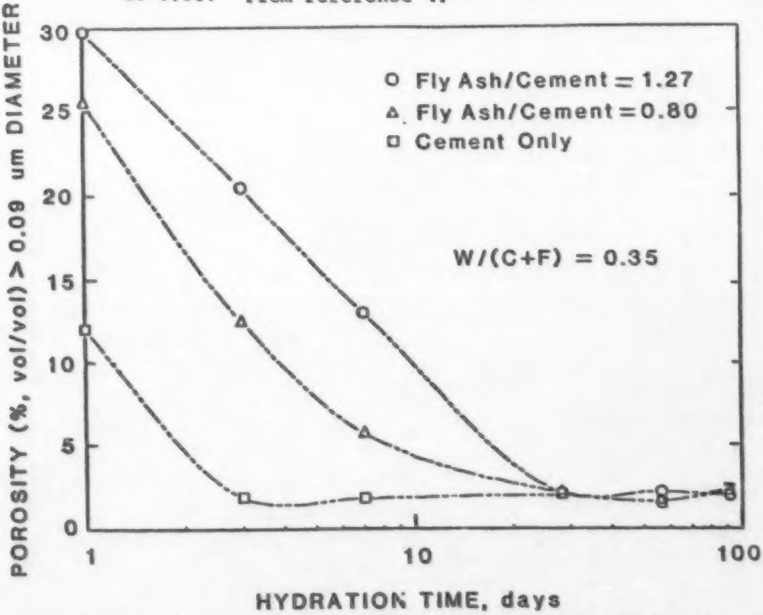

Fig. 2 - Volume of pores (>0.09 μm dia) versus hydration times for pastes with water-to-cementitious material ratio of 0.35. From reference 4.

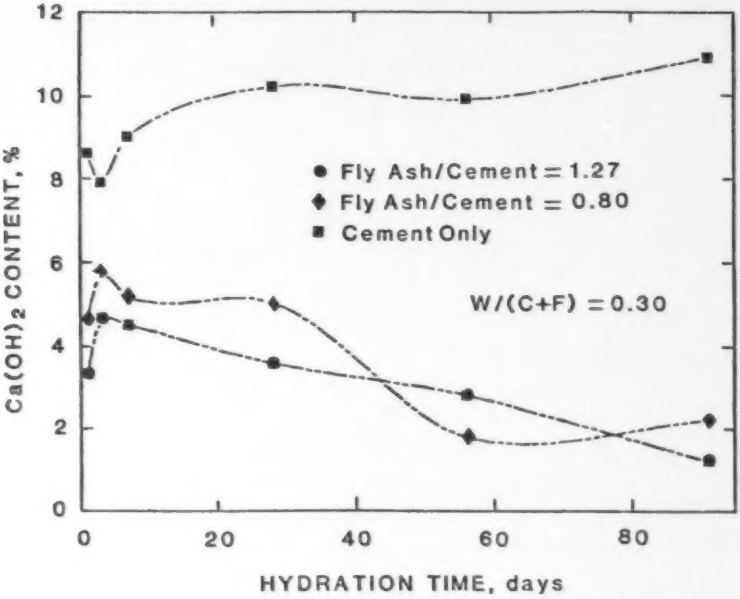

Fig. 3 - Calcium hydroxide content versus hydration times for pastes with water-to-cementitious material ratio of 0.30. From reference 4.

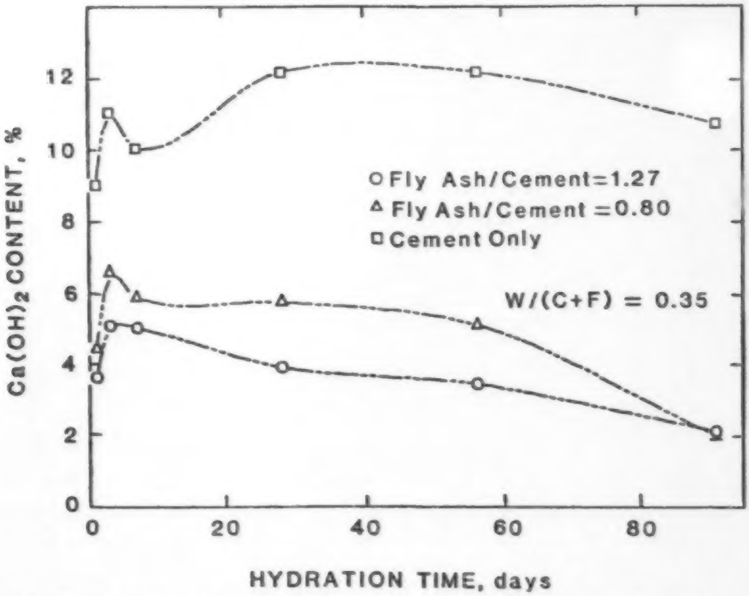

Fig. 4 - Calcium hydroxide content versus hydration times for pastes with water-to-cementitous material of 0.35. From reference 4.

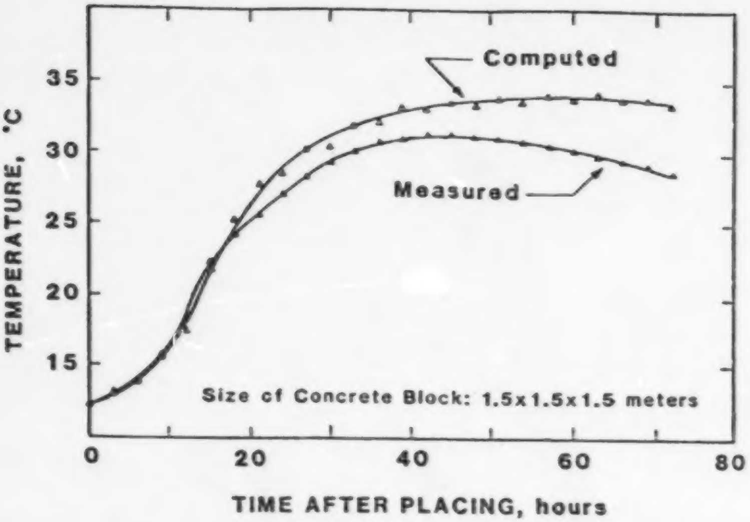

Fig. 5 - Computed and measured temperature with time at mid-height of 1.5 x 1.5 x 1.5-m concrete block.

From reference 12.

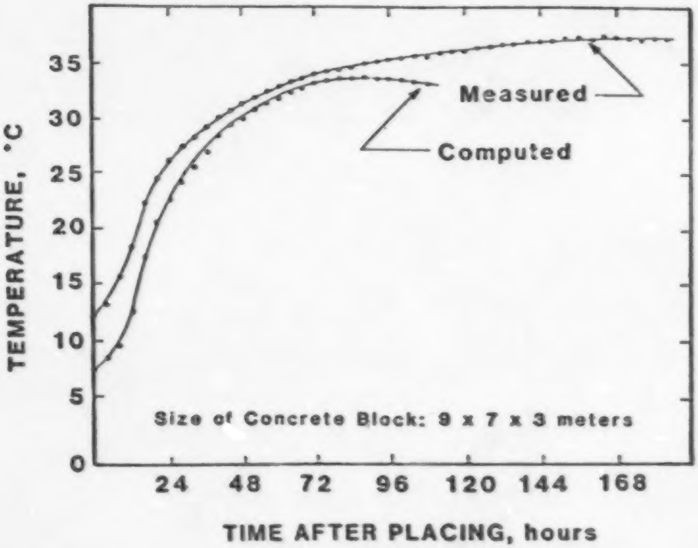

Fig. 6 - Computed and measured temperature with time at mid-depth of 9 x 7 x 3-m concrete block.

From reference 12.

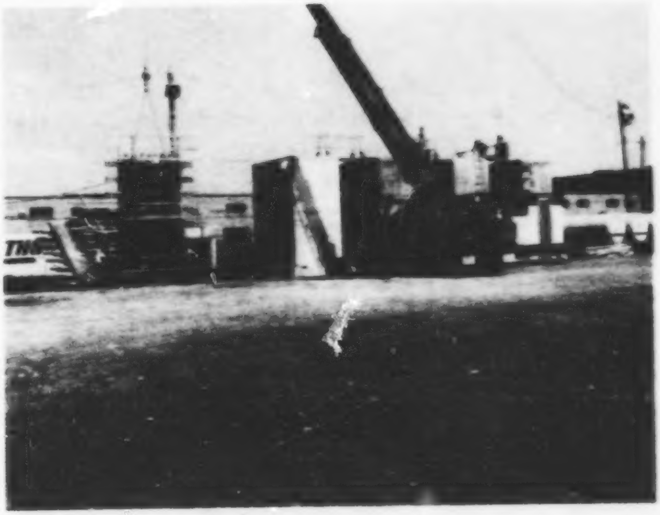

Fig. 7 - A view of 2.5 x 4.0 x 5.0-m concrete blocks for determining rise of temperature and long-term in-situ strength development. From Reference 17.

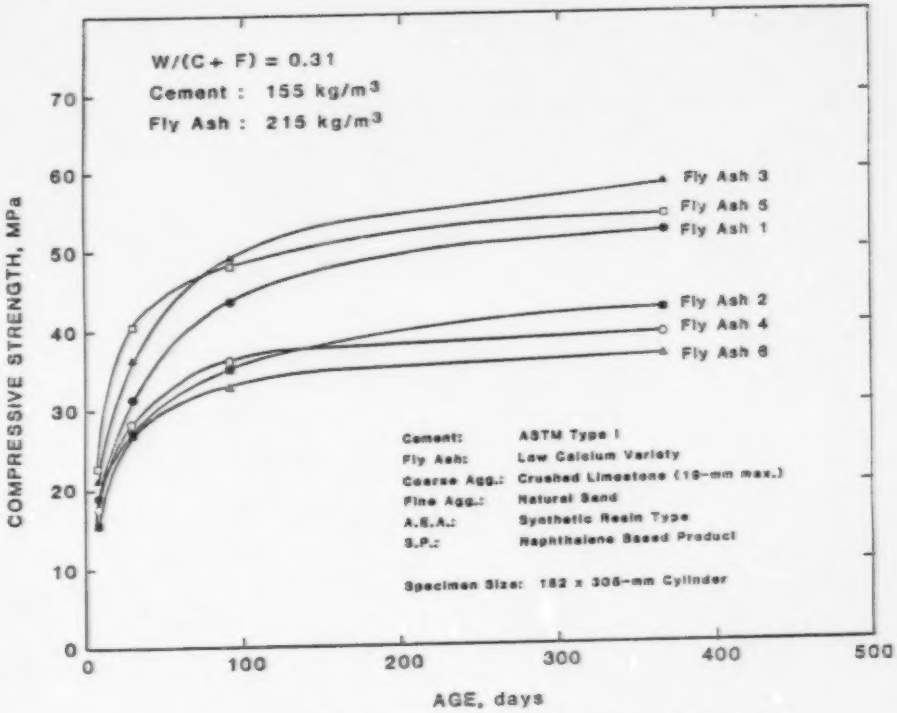

Fig. 8 - Compressive strength development of high-volume fly ash concrete with 155 kg/m^3 of cement.

From reference 14.

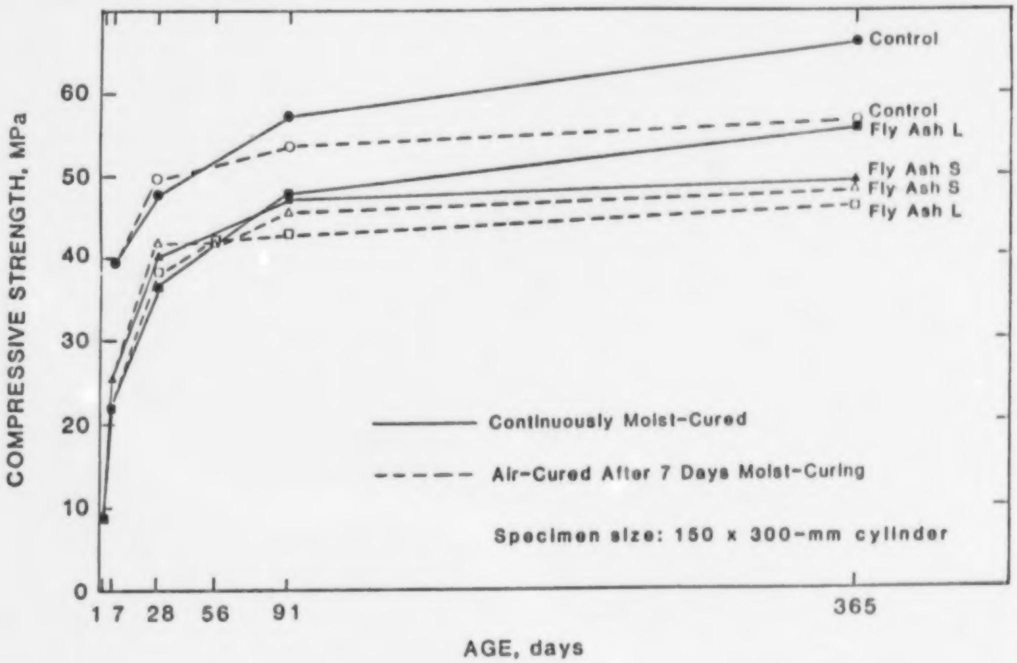

Fig. 9 - Compressive strength development of test cylinders under moist-cured and air-cured conditions.

From reference 12.

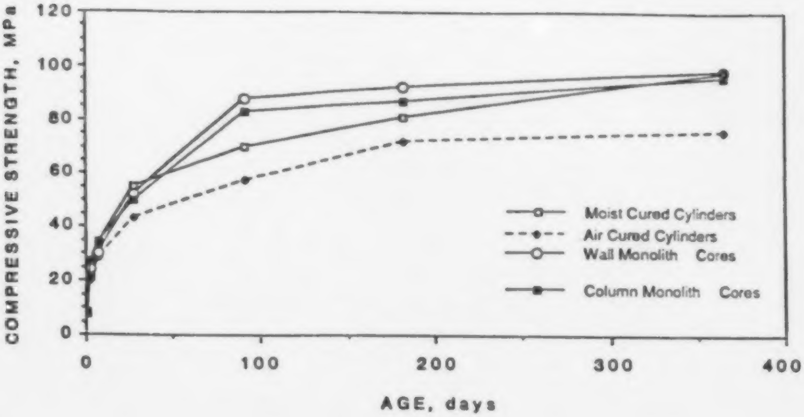

Fig. 10 - Compressive strength development of test cylinders and drilled cored from various types of concrete monoliths made with high-volume fly ash concrete.

From reference 18.

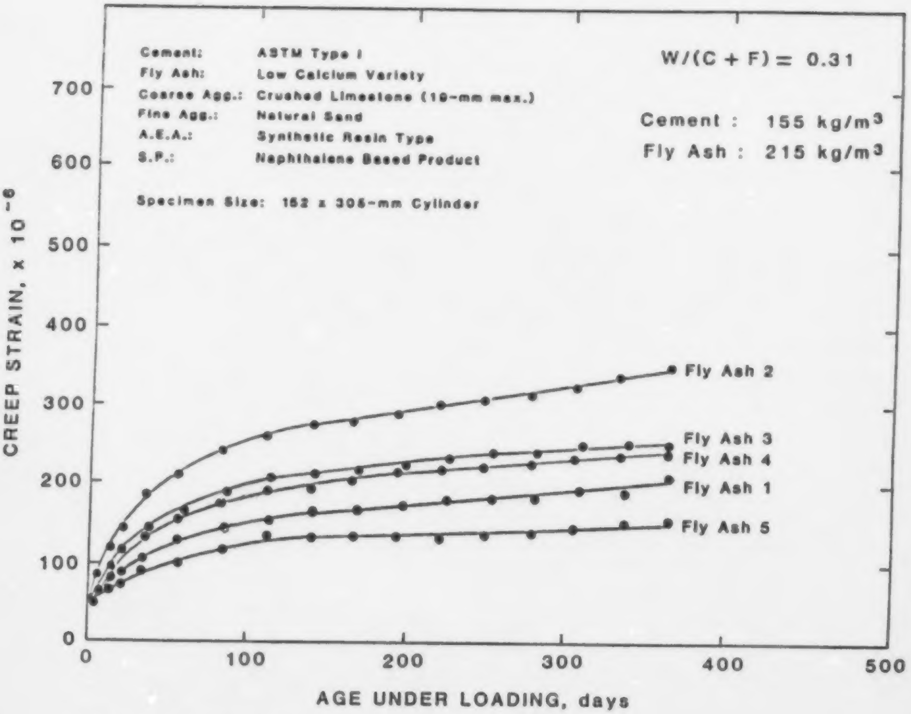

Fig. 11 - Creep strains for high-volume fly ash concretes.

From reference 14.

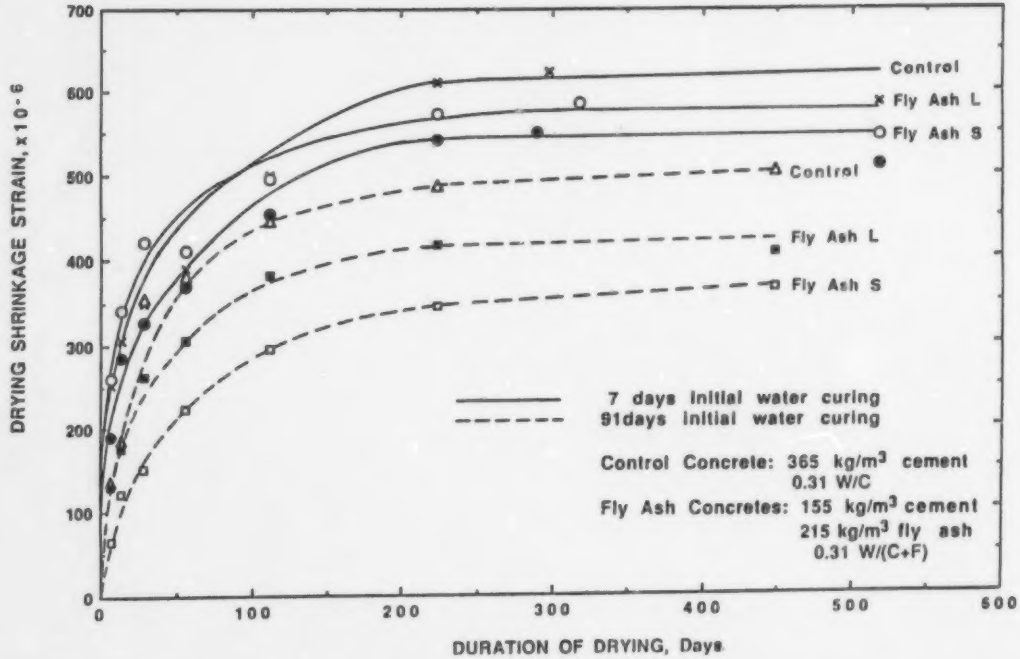

Fig. 12 - Drying shrinkage of high-volume fly ash and control concrete after 7 and 91 days of initial water-curing.

From reference 11.

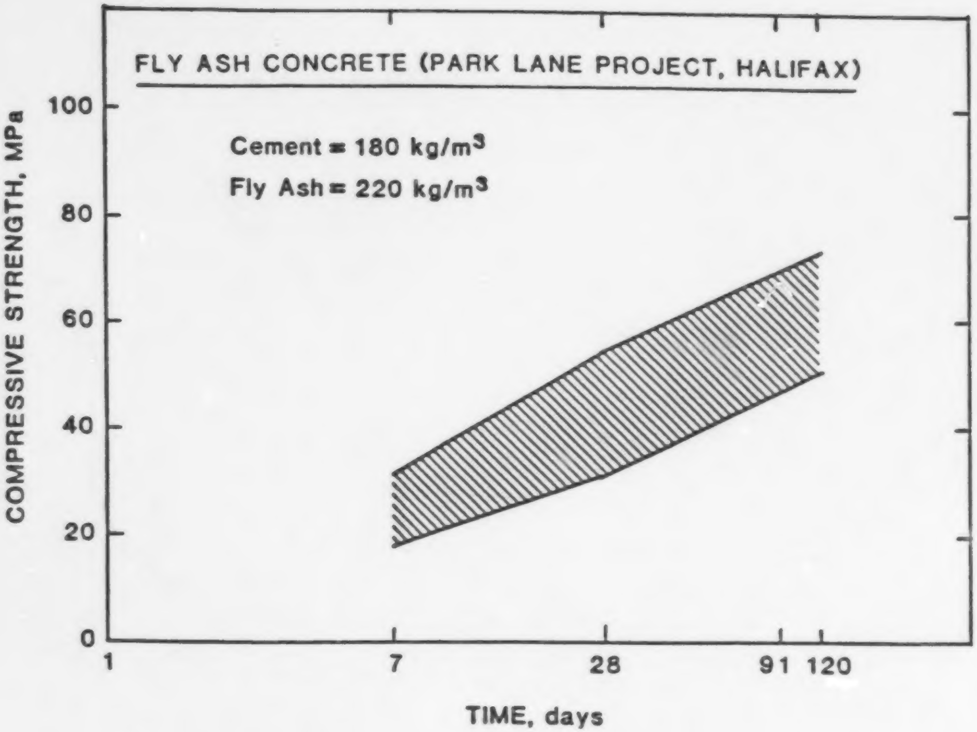

Fig. 13 - Envelope of compressive strength of fly ash concrete in a structural field application in Halifax.

From reference 10.

Chemical Activation of Blast-Furnace Slag: An overview and systematic experimental investigations

by

C. Jolicoeur, M.-A. Simard, T.C. To, J. Sharman, R. Zamojska,
M. Dupuis, N. Spiratos, E. Douglas and V.M. Malhotra

Synopsis: The influence of alkali activators on the hydration rate of blast-furnace (BF) slag has been examined from a review of published data. While a considerable amount of results has been reported on BF slag activation, the generalization and interpretation of the phenomenology observed (rheology, heat evolution, setting time, compressive strength development) is hindered by the lack of extensive data sets, in particular with respect to systematic variations of activator composition. A comprehensive investigation was thus initiated in which paste workability, heat of hydration and compressive strength have been measured for BF slag activated with various chemicals.

The objectives of this work were twofold: 1- to provide the basic data for a broader fundamental understanding of the chemical phenomena involved, and 2- to identify chemical activation systems which lead to high one-day and long-term strengths with adequate workability for periods in excess of one hour. The main chemical activators investigated include: sodium hydroxide, sodium carbonate, sodium silicate of various moduli, and binary or ternary mixtures of these compounds. The effect of other additives, namely, lime, gypsum, sodium sulfate and calcium carbonate, has been tested on some activating systems, as well as the effect of two fluidifying agents: sodium (or calcium) polynaphthalene sulfonate and sodium gluconate. The results clearly illustrate some important generalities, namely, with respect to the role of the carbonate ion in the activation process, the optimal SiO_2/Na_2O modulus in silicate activators and the conditions for increasing workability without undue retardation of setting and strength development.

Keywords: blast-furnace slag; alkali activator; hydroxide; carbonate; silicate; sulfate; lime; gypsum; gluconate; polynaphthalene sulfonate; superplasticizer; workability; hydration; slump; rheology; compressive strength; heat; calorimetry.

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INTRODUCTION

The use of supplementary cementing material, such as silica fume, blast-furnace slag and fly ash, as minerals in concrete has attracted considerable interest in the last two decades. Various beneficial effects resulting from these admixtures have been identified with respect to either the concrete strength or durability, and application codes have been developed specifying the types and contents of desirable mineral admixtures, as well as the procedures for their uses.

Blast-furnace slag (BF slag) exhibit cementitious properties and can be used as hydraulic binders in concrete with or without portland cement. The hydraulic activity of these glassy materials is, however, generally too low for total replacement of portland cement, and chemical "activators" must be introduced to catalyse the hydrolysis/hydration reactions. Strong alkalies (e.g., NaOH) are effective in increasing the hydration rate and hydraulic activity. Incorporation of other alkalies such as sodium carbonate or sodium silicate will help enhance the development of compressive strength to levels comparable to, or higher than, those achieved with normal portland cement. Activated BF slag (with or without portland cement) have thus been well investigated and a number of key results will be outlined in the following section.

General aspects of slag reactivity

The intrinsic hydraulic reactivity of a BF slag depends upon various factors, namely, its glass content, chemical and mineralogical compositions, fineness and age. The observed reactivity in a given system will further depend on the temperature and the chemical composition of the aqueous phase. Many of these factors have been discussed by Hooton (1), Regourd (2), Uchikawa (3), Daimon (4), Smolczyk (5), Von Euw (6), Mehta (7) and Talling and Brandstetr (8). It is frequently difficult to assess the effect of each individual parameter on the reactivity of BF slag; some of the parameters are interdependent and others are frequently unspecified, for example, thermal history, minor components, fineness and particle size distribution, etc. In the following paragraphs, the main influence of several key parameters on slag reactivity as published by various researchers is summarized; in most studies, the slag reactivity was determined from compressive strength measurements.

Glass content and mineralogical composition -- The amorphous (glassy) structure of slags, although difficult to evaluate precisely, is recognized as a major factor of its reactivity (1). Slags having a higher glassy content are generally found to be more reactive. Many authors have observed that the presence of crystallites has a favorable effect on slag reactivity (5-7). These crystallites are formed during the granulation process, and can be different from those formed at equilibrium upon devitrification at high temperature. On the other hand, the relationship between slag reactivity and mineralogical composition has not been clearly established (1).

Chemical composition -- Slags are mainly composed of a silicate network (SiO_2 = S) neutralized by cations such as calcium (CaO = C), magnesium (MgO = M) or aluminium (Al_2O_3 = A). Magnesium and aluminum can also be incorporated into the silicate network (5). Various moduli have been proposed to relate the hydraulic reactivity to the chemical composition (e.g.,: C/S, (C + M)/S, (C + M)/(S + A), etc.) (1,5); correlations between these moduli and the observed reactivity are generally poor, but it seems clear that the reactivity increases with the content of CaO, Al_2O_3 and, to a certain extent, with that of MgO (5).

Fineness, temperature and age -- The hydration of slag particles being a heterogeneous reaction, it is expected, and generally observed, that the reactivity increases with the fineness of the slag. The rheology of fresh pastes is also dependent on the slag fineness.

The reactivity and hydration rate of slag increase with increasing temperature (2); thus, heat curing is often used to enhance reaction rates and strength development. Compared to portland cement pastes, the strength development of slag systems is much more dependent on temperature (3,9).

Finally, with respect to ageing, it has been shown that freshly granulated and ground slags can be sufficiently reactive to be used without chemical activators (3,10). However, the slag reactivity decreases upon standing, through a surface passivation phenomena which remains ill-understood.

Chemical activation of slag

Slag reacts slowly with water to form hydrated products similar to those that are formed during the hydration of portland cement. (5) Hydroxyl ions (OH^-) are known to increase the reaction rate by promoting the solubilization of the aluminate and silicate network (3,8). In this process, the OH^- ions exert a true catalytic function; they are regenerated upon hydrolysis of Si-O-Ca, Si-O-Al, Si-O-Mg, etc. bonds, and will remain available until conditions are appropriate for the formation of weakly soluble hydroxides (e.g., $Ca(OH)_2$, $Mg(OH)_2$). The hydroxyl ions may be introduced from strong alkalies such as NaOH or silicates with low SiO_2/Na_2O modulus, or from weak alkalies such as Na_2CO_3, $Ca(OH)_2$, or silicate with high SiO_2/Na_2O modulus. Alkalies may also be produced through various chemical reactions, for example:

$$Na_2SO_4 + Ca(OH)_2 \rightarrow CaSO_4 + 2NaOH$$
$$C_3S + H_2O \rightarrow CSH + Ca(OH)_2$$

The latter reaction is largely responsible for activation of slags by portland cement. Gypsum, present in the portland cement, can also participate in the activation process (2) as well as carbonate, silicate, sulfate or calcium ions, added from other sources.

Glukhovskii (11) has classified slags as basic, neutral or acid, based on their chemical composition; basic slags can be activated with strong alkalies, with salts of weak acids or with silicates. Neutral or acid slags are best activated by strong alkalies, by silicates or by mixing salts of weak acids with strong alkalies or silicates.

Properties of concretes containing BF slag

Slag mixed with portland cement -- Numerous studies have been reported in which slag is used as partial replacement for portland cement; these studies have been summarized in several review papers (2,3,5,7,12,13). Such a replacement can lead to marked changes on the fresh paste properties, the hydration reactions, and the properties (physical, chemical) of the hardened concretes.

The hydraulic reactivity of slag is generally lower than that of portland cement and, in blended cement, this is reflected by a retardation of the initial and final setting (3,7,12). Curing times have to be increased to allow adequate strength development (12); the setting time is also more dependent on temperature (12).

The early compressive strengths of portland cement-slag concretes are generally lower, as a consequence of the lower hydraulic reactivity; the ultimate long-term strengths, however, are generally higher, mainly due to a more compact matrix with finer pores (3,5,7,12). The evolution of flexural strength follows a similar pattern. Blended portland cement-slag systems usually exhibit a lower permeability to water, alkali and chloride ions; with appropriate air entrainment, equivalent or better freezing and thawing resistances are generally observed (7,13). The shrinkage is, however, significantly higher (7,12).

The lower free lime content of concretes made with slag-blended cements, together with their finer pores, lead to improved chemical stability (13). Slag cement concretes are particularly resistant to sulfates and to sea water (7,13); the alkali aggregate reaction is also greatly reduced (2,3,5,7,13). The less alkaline material is, however, more sensitive to carbonation under certain conditions (7,13).

The observations noted above represent "general trends"; specific behaviors will depend on slag/portland cement ratio, slag properties (chemical composition, glass content, fineness, etc.), or water to binder ratio. In general, and neglecting economic aspects, the motivation for replacement of portland cement with BF slags is to reduce the peak temperatures and to improve the chemical resistance (in particular to sulfate attack) of the concrete.

Alkali activated BF slag concretes -- As noted earlier, mortars and concretes can be made using only alkali-activated BF slag as the cementitious component. The properties of these systems (as fresh pastes or hardened concretes) can be very different from those of portland cement systems and will depend on the physico-chemical features of both the slag and the activator.

Numerous studies have been published on alkali activated BF slags, many of which have been summarized by Talling and Brandstetr (8). In this section, the main conclusions from a series of published studies on the properties of fresh or hardened alkali-activated BF slag mortars or concretes are presented. The rheology of alkali activated slag systems and the development of strength at early age (0-24 hrs) are strongly dependent on the activation system used. As relevant to the purpose of our review, the conclusions from previous studies on workability and strength development for various activated slag systems have been collected and are reported, with references, in Table 1 for several common activators. For compressive strength data, only results for samples cured at room temperature have been considered.

Beyond variations in results, presumably due to variations in slag composition, significant trends can be pointed out:

- upon activation with NaOH, workability is often low, and strength development is largely independent of activator concentration;
- Na_2CO_3, $Ca(OH)_2$ or Na_2SO_4 alone show only a weak activating effect, but the mixture NaOH + Na_2CO_3 is a very effective activator;
- in mixtures containing silicates, the workability is generally improved; the strength development depends on the modulus as well as on the concentration of the silicate; the early strength is low if no other additives (lime, gypsum, ...) are present; the long-term strength is generally very high.

Other properties of alkali activated slag concretes are similar to those observed in blended portland cement-slag concretes with high slag contents. Compared to portland cement systems, the shrinkage of alkali activated slag concrete is generally more important (14,23) and the rate of carbonation is faster (27); the diffusion rates are lower and the chemical resistance is generally higher (21,27). Alkali activated slag concretes often exhibit microcracking (27), particularly with some silicate activators (14), and this phenomena has detrimental effects on flexural strengths (27). Alkali activated slag concretes can also be affected by the alkali aggregate reaction, depending on the alkali content of the activator (21).

The present study was undertaken to identify, as clearly as possible, the role of various ionic species (e.g., Ca^{+2}, OH^-, CO_3^-, etc.) in the phenomenology of BF slag activation. The approach adopted consists in studying the properties (workability, hydration rate and strength development) of slag-water pastes in the presence of selected activators. The study was designed to examine, first, the influence of each activator individually and, second, binary and ternary combinations of these activators. Since the workability of activated slag systems is often limited, the effect of several fluidizers (or superplasticizers) were also investigated. The work envisaged required the combined expertise in colloid chemistry (Université de Sherbrooke), slag concrete (CANMET) and superplasticizer chemistry (Handy Chemicals).

MATERIALS AND METHODS

Materials

Cementitious materials -- The granulated ground BF slag investigated was obtained from a plant in Northern Ontario. The chemical composition, as obtained by X-ray fluorescence, of the slag and of a Type-10 portland cement used for comparison is given in Table 2. The XRD spectrum of the slag showed that it is highly amorphous, containing only small quantities of merwinite.

Chemicals -- The activators used in this study were Reagent Grade laboratory chemicals except for industrial grade Type-N sodium silicate (M_S = 3.2), sodium gluconate, tripolyphosphate and nitrilotriacetate. Different moduli of silicate were prepared from silicate "N" and NaOH. Disal and Durasar superplasticizers which are respectively the sodium and calcium salts of polynaphthalene sulfonates (PNS), were supplied by Handy Chemicals Ltd., La Prairie.

Methods

Sample preparation -- All chemicals added in the slag pastes were taken into account in the calculation of water to binder ratios (W/B): the active materials were considered as binder, and the adsorbed or chemically reacted water was included in the total water content (ex.: 1g of $Ca(OH)_2$ = 0.757 g of CaO + 0.243 g of H_2O). The insoluble materials were dry mixed with the slag (unless specified otherwise) and water soluble materials were dissolved in water.

In calculating the alkali content of the activators, the following procedure was followed. For NaOH and NaSil, the total sodium content is given, expressed as weight percent of Na_2O. With other salts, the amount added is quoted as wt %, and their sodium content is not added to that of Na_2O.

Workability measurements: mini-slump test -- Slump measurements were made on BF slag pastes at W/B ratio of 0.40 or 0.50. The paste samples were prepared in a reproducible fashion by rapid hand mixing of the solids (300 grams) and liquids, followed by intense stirring at 25°C during 3 minutes using a hand-held stirrer (Braun, model MR72). Part of the fluid pastes were transferred into a "mini-slump" cone described by Kantro (28): the workability is defined here as the spreading area of the paste on a vibration-free plexiglass surface. The procedure was carried out at several time intervals after initial mixing to obtain the rate of slump loss: the area at the base of the cone was substracted from the spread areas to yield zero slumps in non-flowing systems.

Compressive strength measurements -- Compressive strengths were measured on 25 mm diam. x 50 mm long cylinders. Paste samples were hand-mixed and poured into 25 mm I.D., 65 mm PVC tubes: after one day, or once set, the cylinders were cut at precisely 50 mm using a ceramic tile rotating blade saw. The samples were stored in 100% RH atmosphere at room temperature and tested at 1, 7, 28 and 91 days.

Calorimetry -- The paste sample (100 g) was poured into a polyethylene bottle which was then immersed in 100 g of water contained in a calorimetric vessel (Dewar flask). The latter is surrounded by a water jacket, the temperature of which is forced to follow that of the paste. Pastes of water to binder ratio of 0.40 or 0.50 were prepared by hand mixing for two minutes. The temperature of the paste was monitored for 24 hrs through a data acquisition system.

RESULTS

Workability in the absence of superplasticizers

Slag and cement in water -- The results of slump measurements on (inactivated) slag and T-10 portland cement pastes at W/B ratios of 0.40 and 0.50 are given in Fig. 1. The non-activated slag pastes have comparable rheological behaviors to that of portland cement pastes at the same water to binder ratio. The workability of these systems is very dependent on their water content, and at constant water to binder ratio, the workability is mainly affected by the surface area of the cementitious materials (in this case: 389 and 452 m^2/kg for the cement and the slag respectively).

Workability measurements on slag pastes at W/B ratio of 0.50 have been repeated regularly over a period of 2 months. Results show that the workability at 10 minutes is less reproducible than at 30 or 60 minutes. The mean values and standard deviations of five measurements for the slump area are 100 ± 10 at 10 minutes, 84 ± 4 at 30 minutes and 75 ± 4 at 60 minutes. Measurements carried out at 20 and 30°C showed that the paste workability is not greatly affected in this temperature range.

Slag activated with NaOH -- The workability of slag pastes activated by NaOH at various concentrations and two water to binder ratios is presented in Fig. 2. Compared to slag or portland cement pastes prepared in water, slag pastes containing NaOH exhibit a similar workability at 10 minutes, but a rapid loss of workability as function of time. For both water to binder ratios, an increase in NaOH concentration over 1% Na_2O increases the workability at 30 and 60 minutes, but the latter remains much lower than in water. It was also found that with 0.1% NaOH, the slag paste was much more fluid than in pure water (not shown); this workability is lost gradually up to 1% NaOH, and is probably a result of variations in the zeta potential of the slag when the alkali concentration is too low for significant activation.

Slag activated with NaOH and Na_2CO_3 mixtures -- The addition of Na_2CO_3 (1 or 2%) to slag pastes containing 2% or 4% Na_2O from NaOH has no effect on the workability at 60 minutes, but increases slightly the workability at 10 and 30 minutes; this increase is proportional to the increase of Na_2CO_3 content (not shown).

Slag with various additives -- The effect of various additives on the workability of slag pastes was investigated with or without addition of 0.3% NaOH at a W/B ratio of 0.50; the concentration of these additives was 2%. The results are presented in Fig. 3, showing workability data at 60 minutes. Generally, addition of various salts increased the workability of the slag pastes, both in pure water and in 0.3% NaOH solution, though the increase was more pronounced in the latter. From the data shown here, it is apparent that the workability of slag pastes depends on many factors, and is not easily predicted. The most striking result is the high workability obtained with several sodium salts: tripolyphosphate and gluconate (also citrate and tartrate, not shown), which are effective dispersants and calcium-complexing agents; sodium-NTA, however, also a good complexing agent, exerts a rather weak influence on paste workability.

Slag activated with sodium silicate -- The workability results obtained in the presence of sodium silicate are shown in Fig. 4, where, depending on the silicate modulus (SiO_2/Na_2O), the workability can either be increased or decreased. At modulus lower than 0.50, the workability is low, comparable to that of pastes with NaOH. For M_S between 0.50 and 1.5, the workability is very high and the slump loss as a function of time is minimal. For $M_S = 2$, the workability is lost within a few minutes after mixing. The high workability found for slag pastes in the presence of silicates of intermediate moduli had also been noted previously by other authors (14,23).

Fig. 5 illustrates the results of mini-slump measurements performed at 60 minutes, as a function of silicate modulus, for three concentrations of Na_2O (total). While the shape of the curves are analogous, an increase in the activator concentration leads to a decrease in workability of the paste and shifts the M_S value for maximum workability from ≈ 1 at 2% Na_2O to ≈ 0.50 at 3 or 4% Na_2O.

Fig. 6 shows several 60 minute workability results for slag pastes activated with sodium silicate (2% Na_2O) of varying modulus (0.50, 1.0, 1.5), with and without lime. The addition of lime (dry mixed with the slag, or introduced as a 50 wt % slurry) induces an important loss of workability with sodium silicate of low modulus, but has no effect on systems with a modulus of 1.5. Addition of Na_2CO_3 or $CaCO_3$ to slag pastes containing sodium silicate of modulus 0.50 or 1 has no effect on their workability (results not shown).

Workability in the presence of superplasticizers or fluidizers

Slag in water or NaOH solutions -- The influence of fluidifying agents on slag pastes prepared in water or in a NaOH (2% Na_2O) solution is shown in Fig. 7. In water, the addition of Na-PNS greatly increases the initial paste workability, and the latter decreases very slowly with time. The addition of sodium gluconate (e.g., 0.1%) increases the workability of the paste, which further increases with time.

In NaOH solutions, sodium poly-naphthalene sulfonate (Na-PNS) and sodium gluconate (Na-GLU) produce comparable increases in paste workability. For the former, this workability is rapidly lost with time, while for the latter, this workability increases with time. Similarly, the addition of NaOH to a slag paste containing Na-PNS decreases its workability, but NaOH additions to pastes containing sodium gluconate has the opposite effect. The mode of action of these fluidifying agents is clearly quite different.

Slag activated with NaOH and Na_2CO_3 mixtures -- As shown in Fig. 8, the addition of 0.50 or 1% Na-PNS to slag pastes activated with (NaOH + Na_2CO_3) produces comparable effects: the workability (at 10 min) is increased by a factor of approximately 2.5, and the effect is lost after 60 minutes. In this system, the calcium salt, Ca-PNS, is less effective than Na-PNS. Sodium gluconate is effective at very low concentrations, and workability is retained over extended periods.

Slag activated with sodium silicate -- The effect of fluidifying additives on the mini-slump results for slag pastes activated by silicates of various moduli is illustrated in Fig. 9 (total Na_2O content fixed at 2%). At low silicate modulus (M_S = 0.3), Na-PNS (1%) has little beneficial effect on the workability of the slag paste, while Na-GLU (0.1%) increases the workability significantly both at early and long times. At intermediate moduli (M_S = 0.50-1.5), the workability of the system with Na-PNS or Na-GLU is comparable to that without addition of fluidizers (see Fig. 4) (the good slump retention observed with Na-GLU is frequently associated with its setting retardation effect as discussed in later sections).

Measurements made on similar systems containing 2% lime show that sodium gluconate has a fluidifying effect, but Na-PNS has no effect on the paste workability. This observation is consistent with results reported by Douglas and Brandstetr (24).

Conclusions from investigation on workability --

- Slag pastes activated with NaOH exhibit a rapid loss of workability; the slump loss can be reduced with a variety of dispersants (e.g., phosphates, borates) and water reducers (gluconate, superplasticizers).
- For slag activated with sodium silicate, the paste workability is dependent on the silicate modulus, and on the (added) lime content; with 2% Na_2O (total), high fluidities are found at silicate moduli between 0.50 and 1.5.
- PNS superplasticizers and sodium gluconate are effective fluidifiers in most systems investigated; notably, PNS superplasticizers are not effective in silicate activated slag pastes.

Compressive strength results

Slag in water or in NaOH solutions -- Fig. 10 illustrates results of compressive strength measurements as a function of time for type-10 portland cement and for slag pastes prepared in water or in NaOH solutions at several concentrations. In water, the slag paste develops no one-day strength. In the presence of NaOH, low one-day strengths are observed for Na_2O contents between 0.50 and 5%. The long-term strength exhibits a more complex behavior; it decreases for Na_2O contents in the range 0-1% and then increases with increasing Na_2O concentration. Compared to results for a T-10 portland cement paste obtained in the same conditions (W/B ratio, mixing, curing, etc.), the compressive strengths of the slag pastes are lower at all ages.

Slag activated with NaOH and Na_2CO_3 mixtures -- Compressive strength results for slag pastes activated with mixtures of NaOH and Na_2CO_3 are shown in Fig. 11. In the absence of NaOH, Na_2CO_3 has no significant effect on strength development; however, the mixture NaOH-Na_2CO_3 yields important increases in compressive strength. At a fixed NaOH content (2 or 4% Na_2O), increasing the Na_2CO_3 concentration increases both the early and long-term strengths; on the other hand, at fixed Na_2CO_3 content (1 or 2%), increasing NaOH from 2% to 4% Na_2O generally leads to a decrease in strength. In these mixed hydroxide-carbonate activators, an optimal composition for the strength development is 2% Na_2O (from NaOH) with 2% Na_2CO_3, and it leads to strengths comparable to those observed with portland cement pastes (see Fig. 10). Addition of sodium sulfate to the (NaOH + Na_2CO_3) activator was found to reduce the compressive strengths at all ages (results not shown).

In some tests, Na_2CO_3 was replaced by $CaCO_3$: the latter did not promote strength development over that observed upon activation by NaOH alone, except perhaps at 91 days (Fig. 12).

Slag activated with NaOH and various additives -- Using the same methodology, other tests were carried out on NaOH activation (2% Na_2O) in the presence of several mineral additives, namely $Ca(OH)_2$, Na_2SO_4, $CaSO_4 \cdot 2H_2O$ (Fig. 12). These additives had no significant effect on the early strength development. The long-term strengths (28 and 91 days) are marginally improved, the sulfates being the more effective of the additives examined.

Slag activated with sodium silicate -- Fig. 13(A,B) shows compressive strengths observed with slag pastes containing sodium silicate of various moduli, at two Na_2O contents: 13A: 2%, 13B: 4%. At 2% Na_2O (Fig. 13A), the long-term strength increases with increasing silica modulus, while the one-day strength reaches a maximum at $M_S = 0.3$; no one-day strength is observed for M_S values higher than 0.50 with 2% Na_2O. At 4% Na_2O (Fig. 13B), the strength development as function of silicate modulus is completely different: the early strength increases from $M_S = 0$ to 0.50, and remains relatively constant up to $M_S = 2$. The long-term strength increases with the silicate modulus, but the 91-day strengths appeared somewhat erratic for moduli higher than 1; such irregularities are possibly due to microcracks formation in the samples.

To overcome the problem of low 1-day strength at 2% Na_2O and $M_S > 0.50$ (where the workability is very high), and to avoid using large quantities of activator, we have tested several mixtures of sodium silicate with other additives.

Slag activated with sodium silicate and Na_2CO_3 -- Fig. 14 shows compressive strength results from slag pastes activated by sodium silicate (2% Na_2O) of varying modulus with added Na_2CO_3. At M_S of 0.50, the one-day compressive strength increases with increasing concentration of Na_2CO_3. At higher moduli, no significant one-day compressive strength was observed, with or without sodium carbonate. For all silicate moduli, the addition of sodium carbonate did not improve the long-term strength.

Results obtained at 3% Na_2O (not shown) led to similar conclusions: for example, at $M_S = 0.50$, with 1% Na_2CO_3, one-day strengths greater than 20 MPa were obtained; however, 91-day strengths did not exceeded 30 MPa. For $M_S = 1$, the one-day strengths increased with Na_2CO_3 content, but the long-term strengths remained low.

Slag activated with sodium silicate and lime -- The addition of lime to the silicate activator (Fig. 15) increases the early strength development for M_S values of 1.0 and 1.5. However, for all silicate moduli, the long-term strength is reduced.

In the systems tested here, it seems likely that the reduction in long-term strengths observed with some activators is exacerbated by the small volume of the test samples (25 x 50 mm cylinders). Nonetheless, the observations point to relevant symptoms and potential problems associated with some of the activators, especially when used at large dosages.

Conclusions from investigation on compressive strengths --

- Compared to portland cement pastes, slag pastes in water exhibit no one-day compressive strength, and lower strengths at other ages.
- NaOH activation yields low one-day compressive strengths; up to ≈ 2% Na_2O, the long-term strength is reduced (compared to non-activated paste).
- $Ca(OH)_2$, and Na_2SO_4 or $CaSO_4$ in NaOH-activated slag pastes slightly improve the long-term compressive strengths.
- Na_2CO_3, in the presence of NaOH, greatly increases compressive strengths.
- With sodium silicate at low Na_2O content, the early strength is developed only for $M_S < 0.50$, and is enhanced by the addition of Na_2CO_3; at higher Na_2O contents, early strengths are developed for all moduli.
- Addition of lime to silicate activators promotes the early strength development for $M_S > 0.50$, but may be detrimental to long-term strength.

Calorimetry

Calorimetric experiments were carried out on slag hydration with selected activators. These measurements were designed to obtain data on the reaction kinetics of several systems of basic interest and on systems which exhibit promising rheological or strength properties. The results are summarized in Fig. 16.

Slag mixed with water (curve A) will start to react after a lag period of approximately 20 hours, with little amounts of heat evolved. In the presence of NaOH (2% Na_2O) (curve B), the lag time is reduced to zero, and the amount of heat generated is increased. The addition of 2% Na_2CO_3 (curve C) to the last system further increases the amount of heat without appearing to affect the lag time. The addition of Na-PNS to this system does not appear to affect the evolution of heat (results not shown); while the addition of sodium gluconate (curve D) increases the lag time substantially, but the heat generation is only marginally affected.

The thermogram of slag activated with sodium silicate, 2% Na_2O, M_S = 0.50 (curve E) shows a dormant period of approximately 6 hours and a temperature increase similar to that of slag activated with NaOH. The addition of 2% Na_2CO_3 to the last system (curve F) increases the amount of heat evolved, but has no major effect on the lag period.

Conclusions based on calorimetric investigations --

- BF slag in water suspension reacts slowly after a lag period of ≈ 20 hours.
- Addition of NaOH or sodium silicate increases the reaction rate; the lag period is eliminated with NaOH (2% Na_2O) and is reduced to ≈ 6 hours with Na-Sil (M_S = 0.50, Na_2O = 2%).
- Addition of Na_2CO_3 to slag containing NaOH or Na-Sil increases the reaction rate but has no effect on the lag period.
- PNS superplasticizers do not affect the reaction kinetics, while sodium gluconate has a retarding effect (increases the lag period).

CONCLUDING REMARKS

The systematic study of simple BF slag systems provides a reasonably comprehensive view of the phenomenology of chemical activation of slag. The properties of slag pastes investigated (mini-slump, compressive strength and hydration rate) while varying activation composition (single, binary, ternary systems) and concentrations define useful ranges of the activation parameters for adequate workability and strength development. Ongoing studies on the evolution of the interstitial liquid phase and solids through chemical and microstructural analyses will enable a better understanding of the observations reported here, in particular with respect to the important role of carbonates in strength development and the fluidifying effect of superplasticizers. Further work on BF slag systems will also address the reduction of alkali metal content in chemical activators to minimize the potential for alkali-aggregate reactions and the problem of microcracking observed in some of the systems studied here. Likewise, the various aspects related to durability (permeability, corrosion, freeze-thaw) will also be investigated.

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Table 1 - Summary of reported observations in alkali-activated slag (AAS) systems

Authors	System	Activator concentration	Observations
NaOH			
Anderson and Gram (14)	mortar	3-11%	Lower setting time and higher workability as compared to normal portland cement mortar; 1-day strength is acceptable; strength development is independent of activator concentration and lower than for normal portland cement mortar.
Talling (15)	mortar	3.75-7.5%	Strength development is dependent on slag fineness but not on NaOH content.
Tashiro and Yoshimoto (16)	mortar^1	0.1-2%	Low NaOH concentrations (0.1%) increase the compressive strength while higher concentration reduce it.
Parameswaran and Chatterjee (17)	mortar^2 concrete	2-5%	Stiffening within 10 min; strength increases with increasing NaOH concentration; higher early strength and lower long-term strength compared to portland cement concrete.
Narang and Chopra (18)	mortar	1-5%	Setting time range within the limits specified for normal portland cement; strength very dependent on slag composition.
Sersale *et al.* (19)	mortar	0-15%	Strength increases with NaOH content up to ≈ 8%; low 3-day and 28-day strength.
Metso and Kajaus (10)	mortar	1-4%	Very good activation effect.
Na_2CO_3			
Anderson and Gram (14)	mortar	3-11%	Setting time not dependent on activator concentration; more fluid than slag NaOH mortars; no 1-day strength; long-term strength comparable or higher than that with normal portland cement mortar.

Table 1 - Summary of reported observations in alkali-activated slag (AAS) systems, cont'd

Authors	System	Activator concentration	Observations
Tashiro and Yoshimoto (16)	mortar[1]	0.1-2%	Compressive strength is reduced at all ages, except for 0.1% Na_2CO_3 where it is comparable to mortar without additive.
Parameswaran and Chatterjee (17)	mortar[2]	N/A	No 1-day strength and low 28-day strength.
Narang and Chopra (18)	mortar	5%	No strength development.
Sersale *et al.* (19)	mortar	2-10%	Strength increases with increasing activator concentration; strength is low but higher than with NaOH.
Metso and Kajaus (10)	mortar		Poor action effect.
$NaOH + Na_2CO_3$			
Gjorv (20)	concrete		Compared to normal portland cement concrete, viscosity is much higher and yield shear strength is lower; 1-day strength is lower and long-term strength can be higher, if fineness of the slag is high.
Kukko and Mannonen (21)	concrete		Thixotropic mixture; in flow test, spreading continues for a longer time than with normal portland cement mix; water to binder ratio can be very low; high long-term strength.
Metso and Kajaus (10)	mortar		Very good activation effect.
$Ca(OH)_2$			
Parameswaran and Chatterjee (17)	mortar[2]	N/A	No strength up to 28 days.

Table 1 - Summary of reported observations in alkali-activated slag (AAS) systems, cont'd

Authors	System	Activator concentration	Observations
Wu et al. (22)	paste	10.6%	Low activating effect.
Sersale et al. (19)	mortar	0-15%	Low activating effect.
Metso and Kajaus (10)	mortar	1-6%	Moderate activation effect.
Na_2SO_4			
Tashiro and Yoshimoto (16)	mortar[1]	0.25-2%	Generally, early strength is increased and long-term strength is decreased.
Parameswaran and Chatterjee (17)	mortar[2]	N/A	No 1-day strength; reasonable 28-day strength.
Wu et al. (22)	paste	4%	Best activation effect at early age.
Na-Sil			
Anderson and Gram (14)	mortar	M_s 0.9-3.3 4-8%	Setting time increases with increasing silicate modulus; more fluid than slag-NaOH mortar; no 1-day strength; long-term strength is low for modulus of 3.33 but is higher than that of normal portland cement mortar for moduli between 0.9 and 1.8.
Voinovitch et al. (23)	mortar concrete	M_s 1-3.8 2-10%	Na-Sil confers high workability to the mixes; strength development comparable to normal portland cement concrete is obtained with 9% Na-Sil (M_s=1); addition of gypsum increase the strength.
Douglas and Brandstetr (24)	mortar	M_s 2.5 15-25%	Low early strength, but high long-term strength; addition of sodium sulfate gives 1-day strength similar to that of a normal portland cement mortar.

Table 1 - Summary of reported observations in alkali-activated slag (AAS) systems, cont'd

Authors	System	Activator concentration	Observations
Douglas *et al.* (25)	concrete	$M_s \approx 1.4$ 10-15% (+ lime)	Good workability can be obtained; 1-day and 28-day compressive strengths range respectively: 20-39 MPa and 45-60 MPa.
Talling (26)	mortar	M_s 1 4-12%	No 1-day strength is developed for concentration lower than 4%; with 6% Na-Sil, high strengths are obtained.
Metso and Kajaus (10)	mortar	N/A	Poor activation effect.

[1] slag with 52% CaO

[2] slag with ≈ 20% Al_2O_3

Table 2 - Chemical composition (wt %) and surface area of the slag and the cement used in this work

	BF slag wt %	portland cement wt %
SiO_2	37.5	20.7
CaO	30.9	62.3
MgO	19.1	1.6
Al_2O_3	8.2	4.7
Fe_2O_3	0.32	2.8
Na_2O	0.10	0.33
K_2O	0.27	0.96
SO_3	1.5	3.2
Blaine (m^2/kg)	452	389

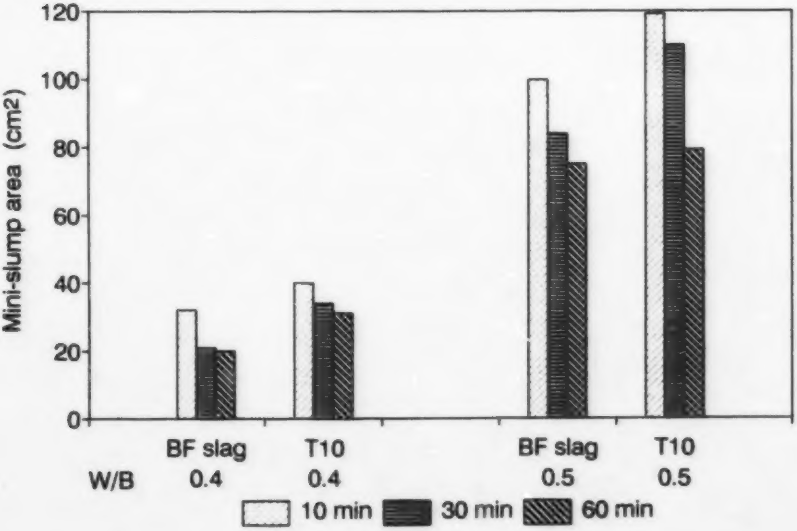

Figure 1: Mini-slump areas of BF slag and cement pastes at W/B ratios of 0.40 and 0.50.

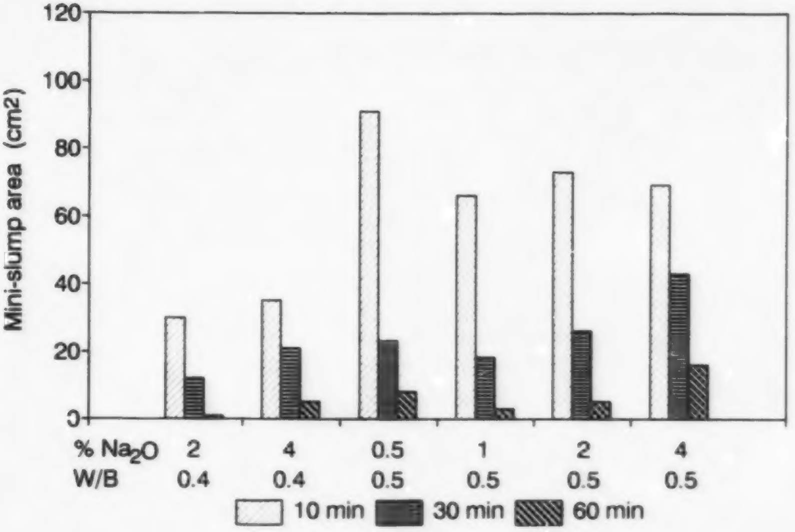

Figure 2: Mini-slump areas of BF slag pastes with various amounts of NaOH added at W/B ratios of 0.40 and 0.50.

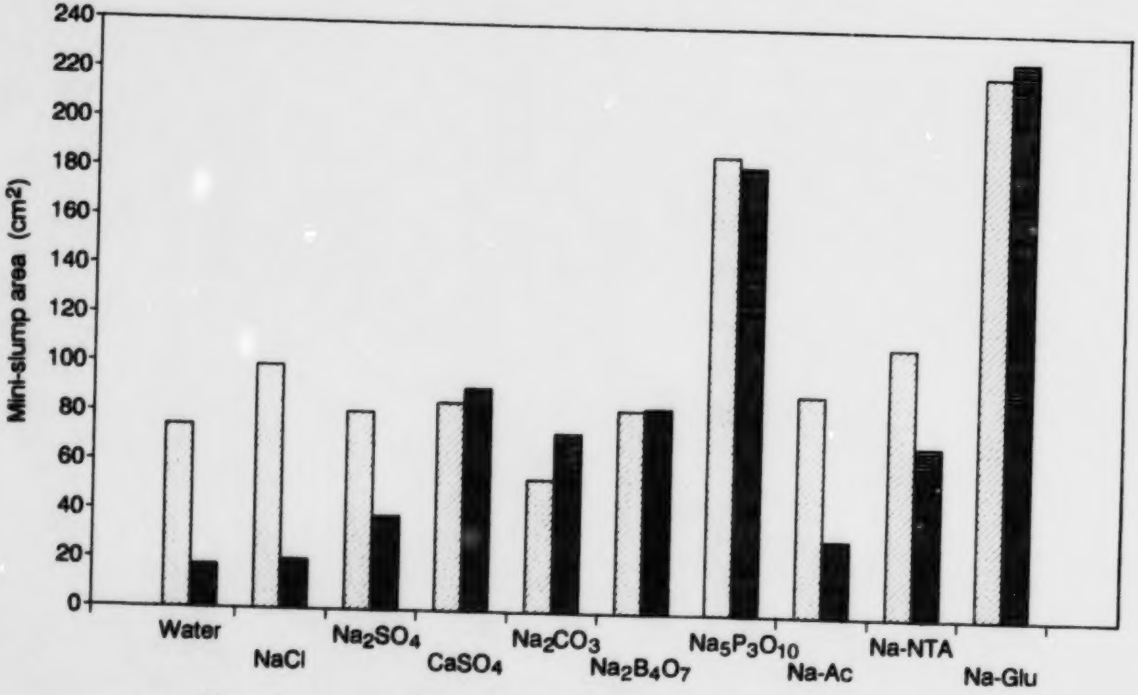

Figure 3: Mini-slump areas at 60 minutes of BF slag pastes with various additives (2% by weight), at W/B ratio of 0.50; in pure water: first bar; with 0.3% NaOH: second bar.

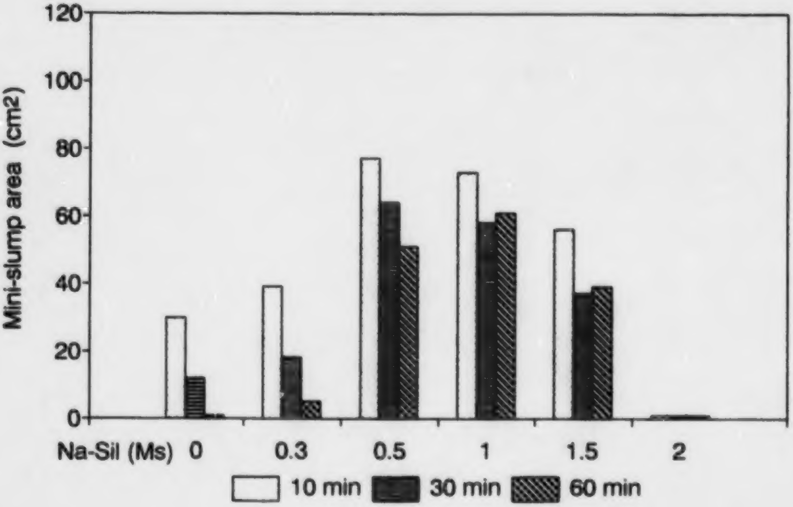

Figure 4: Mini-slump areas of BF slag pastes, W/B ratio of 0.40, activated with sodium silicate of various SiO_2/Na_2O ratios.

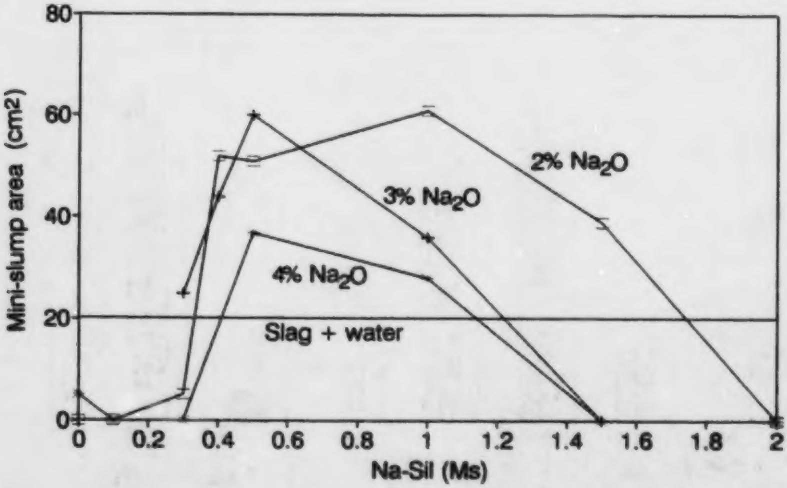

Figure 5: Mini-slump areas at 60 minutes of BF slag pastes, W/B ratio of 0.40, activated with sodium silicate of various moduli, at three Na_2O concentrations.

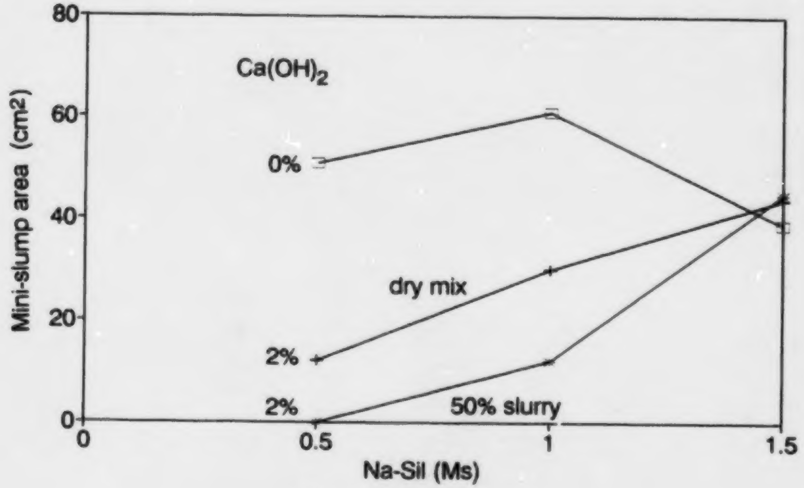

Figure 6: Effect of lime on mini-slump areas at 60 minutes of BF slag pastes, W/B ratio of 0.40, activated with sodium silicate of varying modulus at 2% Na_2O.

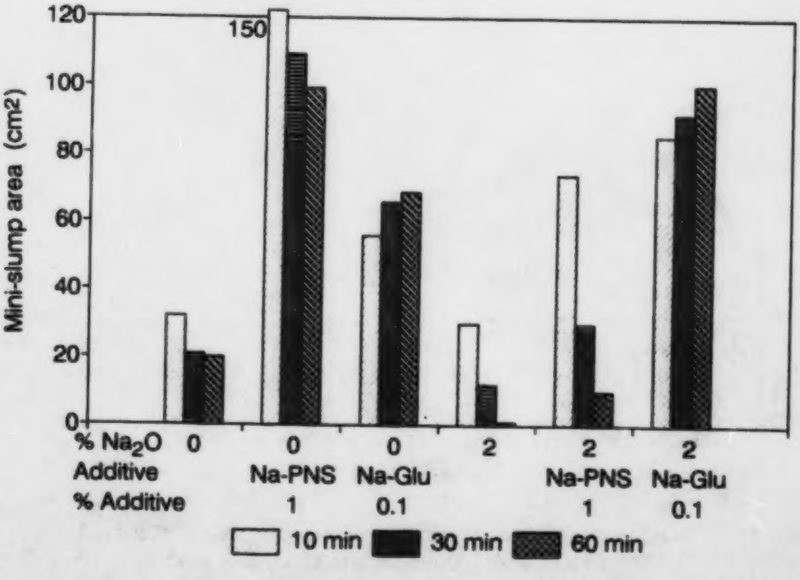

Figure 7: Effect of fluidifying agents on the mini-slump areas of BF slag pastes, W/B ratio of 0.40, prepared in pure water or in NaOH (2% Na_2O).

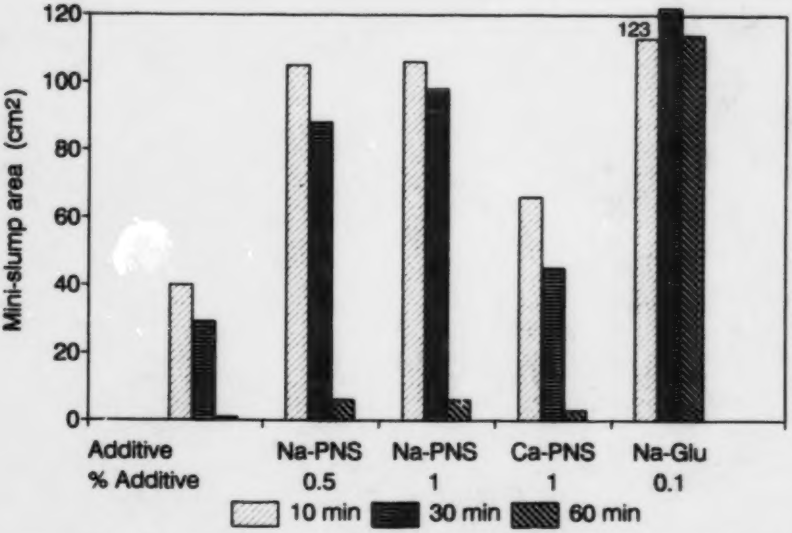

Figure 8: Effect of fluidifying agents on the mini-slump areas of BF slag pastes, W/B ratio of 0.40, activated with a mixture of NaOH (2% Na_2O) and 2% Na_2CO_3.

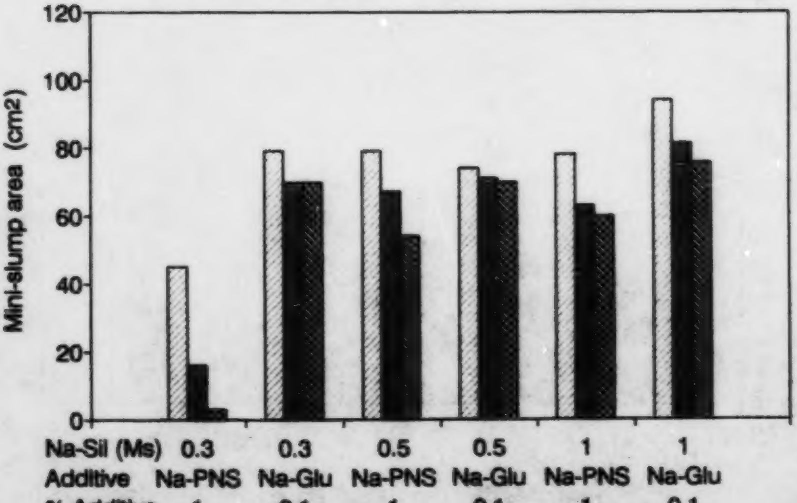

Figure 9: Mini-slump areas of BF slag pastes, W/B ratio of 0.40, activated with sodium silicate of varying modulus (2% Na_2O), with fluidifying agents.

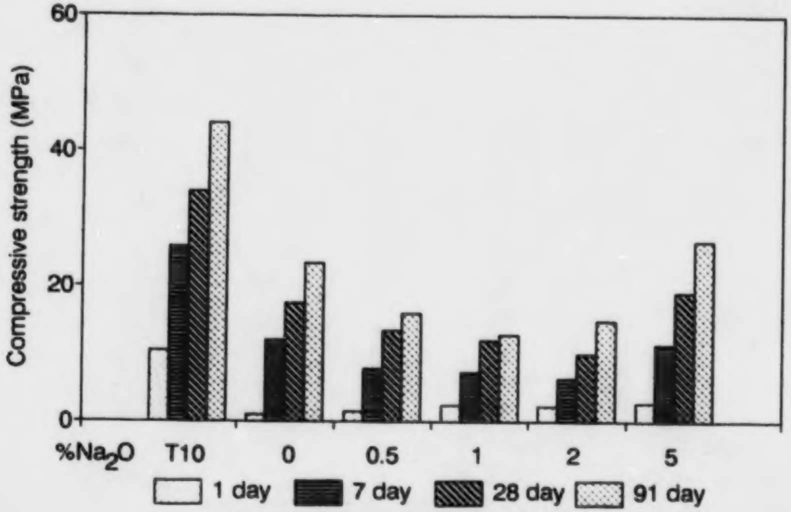

Figure 10: Compressive strengths of Type-10 portland cement and of BF slags activated with NaOH of various concentrations, at W/B ratio of 0.40.

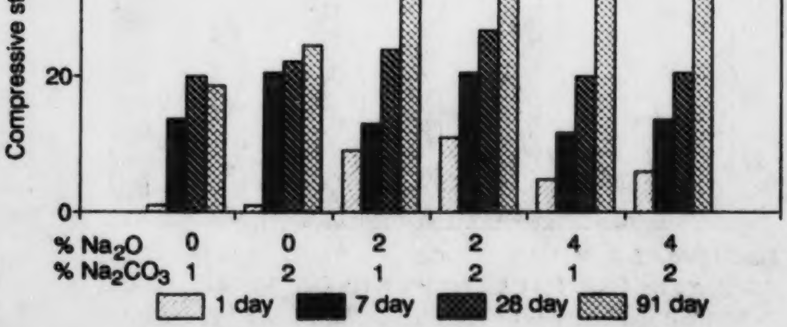

Figure 11: Compressive strengths of BF slag activated with mixtures of NaOH and Na_2CO_3, at W/B ratio of 0.40.

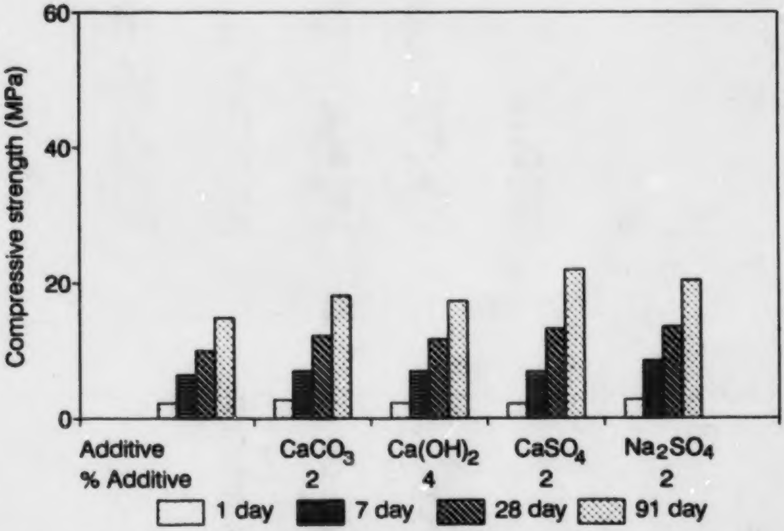

Figure 12: Compressive strengths of BF slag activated with mixtures of NaOH (2% Na_2O) and various additives, at W/B ratio of 0.40.

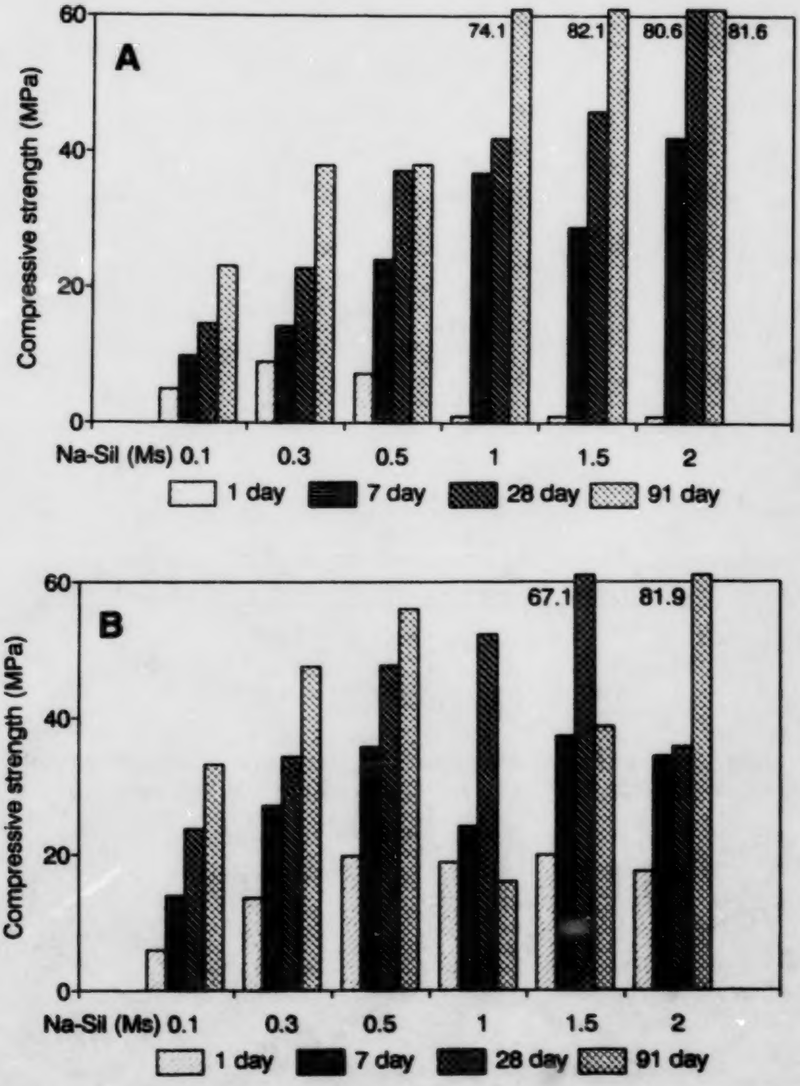

Figure 13: Compressive strengths of BF slag activated with sodium silicate of various moduli, at W/B ratio of 0.40; A: 2% Na_2O; B: 4% Na_2O.

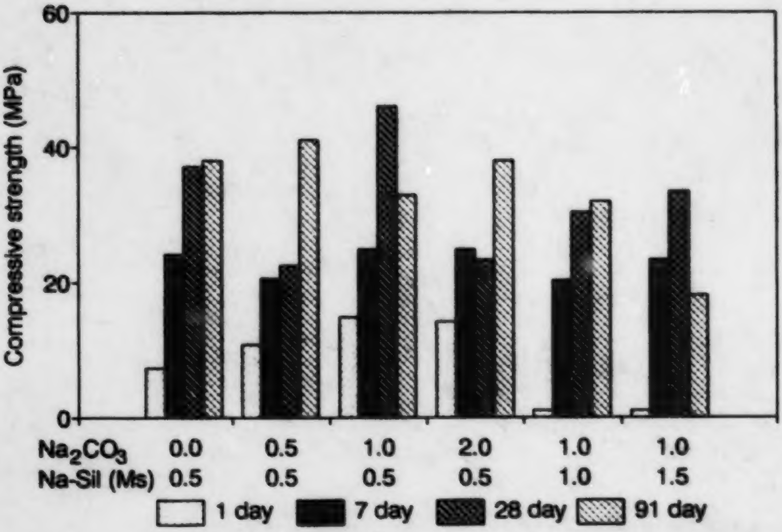

Figure 14: Compressive strengths of BF slag activated with mixtures of sodium silicate (2% Na_2O) of various moduli and Na_2CO_3, at W/B ratio of 0.40.

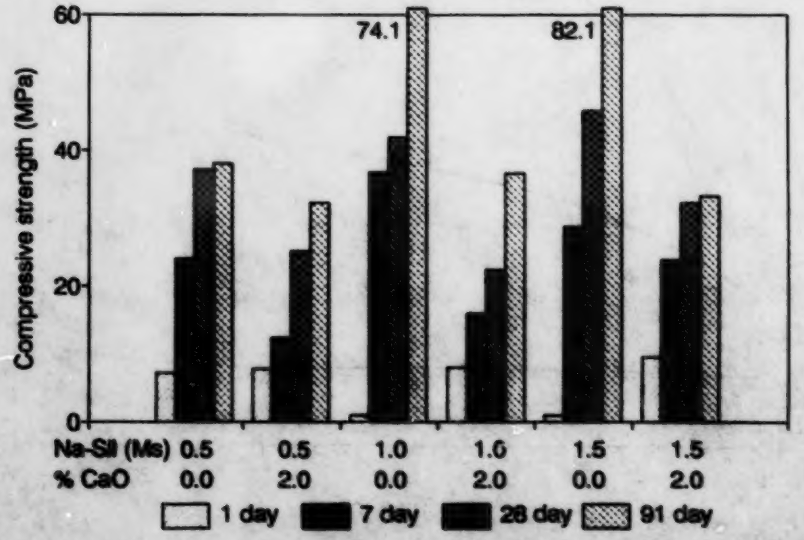

Figure 15: Compressive strengths of BF slag containing lime, activated with sodium silicate of various moduli at a concentration of 2% Na_2O and at W/B ratio of 0.40.

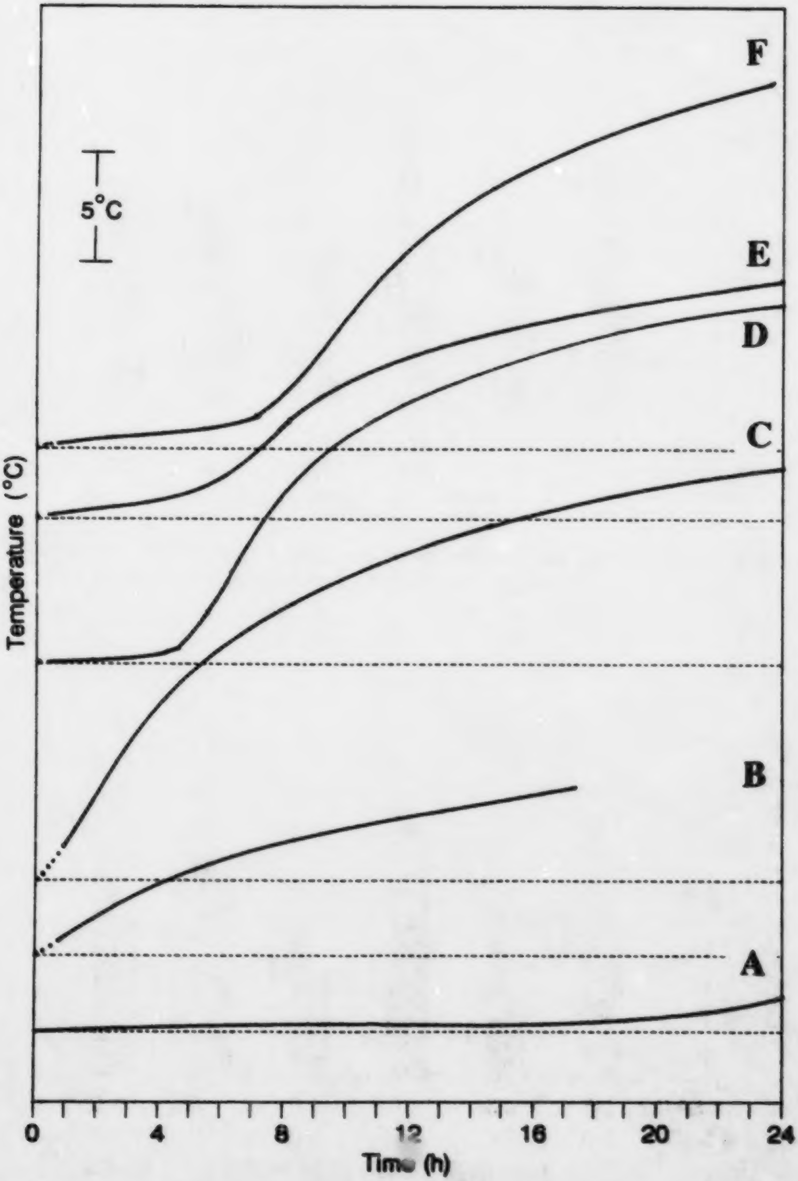

Figure 16: Thermograms of BF slag activated with various activator systems, at W/B ratio of 0.40.
A: Water; B: NaOH (2% Na_2O); C: NaOH (2% Na_2O) + 2% Na_2CO_3; D: C + 0.2% sodium gluconate; E: sodium silicate Na_2O = 2%, M_s = 0.50; F: E + 2% Na_2CO_3.

Concrete Durability - Recent Solutions to Enduring Problems

Harold Roper and Daksh Baweja

Synopsis: Recent advances in the field of concrete durability are presented. The subject is considered within the main areas of durability as outlined by the American Concrete Institute. Reference is made to durability requirements relating to field performance of concrete. A search of the recent literature allows the choice of references on areas of advance in durability studies. If there is a general desire for durable structures, greater attention will need to be focused on potential causes of such problems and how to minimise them in each case. In order to achieve this, a knowledge of the most recent advances must be transferred to practice.

Keywords: Deterioration Durability Corrosion Alkali-Aggregate Reaction Sulphates Chlorides Carbonation Fly Ash Slag Silica Fume

Harold Roper, Associate Professor in the School of Civil and Mining Engineering, The University of Sydney, has teaching, research and consulting interest in the field of materials for use in reinforced and prestressed concrete structures. He has thirty years experience in cement and concrete technology and is the author of over seventy research papers on the properties of aggregates, cements and concretes.

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PREAMBLE

Recently, the senior author (HR) was approached by a structural consultant with the request that he advise on techniques and materials to provide acceptable durability to a concrete flue for gas emission from a manufacturing plant. The temperature conditions were to be variable, the gas moist, the emissions sulphurous and exposure would be considered generally adverse. The structural consultant ruled out the use of linings, as he argued that replacement thereof would be difficult. The advice given to him was to incorporate either steel fibre reinforcement or a light mesh reinforcement to control surface crack size, to use silica fume as a pozzolanic material, to reduce the water:cement ratio to less than 0.35 by the incorporation of a superplasticising admixture in addition to applying a series of other measures.

After pricing each requirement, the structural consultant decided that his original cost estimates would not allow their adoption. A conventionally reinforced, non-superplasticised fly ash mix was eventually specified by him and used for financial reasons. After a relatively short period of operation, no sign of durability problems have developed within the flue, however, this experience highlights three important procedural aspects which need to be adopted if durability is of critical importance in the function of a structure. These are:-

1. As ensured durability comes at an initial higher cost, this must be allowed for in any construction planning.
2. Time and attention should be given to the proper selection of materials at the design stage in the same manner as these are devoted to structural considerations.
3. If special testing of mixture proportions, procedures or even prototype sections are called for, sufficient scope and time should be available to the concrete technologist to assess the results before final decisions are made.

The scope, planning and procurement of test data will vary from project to project, but if there is a general desire for durable structures, greater attention will need to be focused on potential causes of such problems and how to minimise them in each case. The understanding of potential causes of problems requires basic research on the one hand, and examination of structures and their environment on the other. Minimization requires application of the information gleaned from basic studies. This may lead to the development of new materials, or modification of traditional ones. New or improved techniques for their use may emerge. Another aspect of problem minimization is the requirement for diagnostic test procedures, which allow reliable prediction of deterioration or durability effects on real structural elements. This pursuit may lead to the development of predictive modelling of deterioration processes under service conditions. To check that the objective of improved durability has been achieved, surveys of in situ structures are once again required. The authors in addressing the subject of durability, will attempt to demonstrate some important interactions of each of these processes.

INTRODUCTION

Definitions

In introducing the subject of durability, it is essential to attempt a definition thereof. The Shorter Oxford English Dictionary defines durability as "the quality of being able to withstand change, decay or wear". In applying such a definition to concrete, it must be appreciated that no concrete yet manufactured will continue to fulfil its role indefinitely, and that the same concrete may perform perfectly over a long period in one environment while deteriorating relatively rapidly in another. As an example of this, consider the performance of a high C_3A-cement concrete exposed either to dry air or in contact with sulphate charged groundwater. Although it is expected that such concrete would be less durable under the second condition, Cohen and Mather (1) point out the problems of performance prediction in the presence of sulphates.

Rather than consider the dictionary definition, one may seek a definition of durability and its related concept, serviceability, in technical literature related to concrete. In this case we find the following:-

Durability	The ability of concrete to resist weathering action, chemical attack, abrasion and other conditions of service. (From Cement and Concrete Terminology - ACI 116R-85).
Durability	The ability of concrete to resist weathering action, chemical attack, abrasion or any other process of deterioration. (From Guide to Durable Concrete - ACI 201.2R-77).
Durability	The safe performance of a structure or a portion of a structure for the designed life expectancy. (Note - Because the forces of nature coupled with some man-created conditions of exposure, may cause progressive deteriorations, these recommendations do not preclude the need for normal maintenance). (From ASTM Recommended Practice for Increasing Durability of Building Constructions Against Water-Induced Damage - E 241-77).

Durability	The capability of maintaining the serviceability of a product, component, assembly or construction over a specified time. (From ASTM Recommended Practice for Developing Accelerated Tests to Aid Prediction of the Service Life of Building Components and Materials E 632-82)
Durability	The durability of concrete, (in particular, resistance to abrasion, sulphate attack and water penetration) shall be suitable for the expected service conditions. (Note - Where service conditions include exposure to the weather, particularly in close proximity to the coast, durability problems have been encountered in recent structures. In part, the problems have arisen because modern concretes have relatively higher permeability and thus offer less protection to reinforcement. Poor construction practice and the use of admixtures containing calcium chloride have also been associated with some failures). (From Australian Standard AS 1480-1982).
Serviceability	The capability of a building product, component, assembly or construction to perform the function(s) for which it is designed and constructed. (From ASTM Recommended Practice for Developing Accelerated Tests to Aid Prediction of the Service Life of Building Components and Materials E 632-82).

Certain of these definitions are broad, almost all embracing. Of greater interest, others highlight the fact that time plays an important role in the durability of concrete. Furthermore, some indicate that durability is to be considered at the design stage of a project, and that normal maintenance may, or may not be allowed for in reviewing durability.

Quantification of Durability In Service

As yet, no one has satisfactorily quantified the durability of concrete in service, although some attempts at such evaluations have been based on statistical concepts (2,3,4). Durability analyses, if statistically considered, have to include both the possibility of certain events taking place, and the possible consequences thereof (5,6). This leads to complicated risk analysis procedures which are only really worthwhile if the damage mechanisms can be described reasonably well. This stage has not yet been reached even for thoroughly studied phenomena such as sulphate attack, where a change in the cation species without changes in the sulphate concentration may entirely upset any prediction of the rate of attack on concrete. This type of difficulty has been highlighted in the paper by Cohen and Mather (1).

Some rating systems for the evaluation of field durability problems have been described in the literature. There are many examples of computerised rating systems for the evaluation of the in situ condition of bridges, one such by Miyamoto et Al (7) is a knowledge based system which incorporates the concepts of basic probability to deal with subjective information related to bridge rating. Rating systems are also available for buildings. A rating system applied by Sabnis, Sorokko and Doshi (8) on a rehabilitation of Grand Central Station in New York City has been described. Based on their restoration and rehabilitation experiences with bridges and other buildings, they recommend application of such computerized rating systems for in situ assessment.

Work by the authors and co-workers on the development of a quantitative measure of concrete durability, first proposed in 1984 (9), was recently used, in part, to evaluate the field performance of building structures in coastal and adjacent areas of selected suburbs in Sydney, Australia (2). Field performance of this set of building structures was assessed solely on the basis of observed reinforcement corrosion on concrete structural elements. Such studies highlighted the difficulties in attempting to classify all types of in situ deterioration effects. For any given concrete structural element, a total of up to fifty different deterioration effects could be found, thus creating problems of accurate data collection, storage, retrieval and analysis (10). A list of the deterioration effects is presented in Table 1. It can be seen that by combining certain of these, the number of effects can be appreciably reduced, however, specific causal relationships may not now be ascribed to these.

Models and Their Application

One approach adopted by practitioners is to use information on the physico-chemical phenomena to formulate models which portray the actions of corrosion, carbonation or other aggressive conditions on structural members. Rigourous mathematical models such as those of Bazant for corrosion (11,12) have not been generally embraced since local features perturb the results significantly, and modelling these as opposed to the global properties of the concrete is extremely difficult. For that reason, simpler models such as those of Tutti (13), Beeby (14) and Browne (15,16) are often used. Most of the models so far developed and applied are based on diffusion theory but as even Browne has noted, it is not yet established that a simple diffusion law can be used for chloride ingress with time.

The phenomenon of carbonation has often been modelled as a simple diffusion process (17). Verification of such equations under certain service conditions has been sought (18,19,20,21). However rates of carbonation in structures are very variable, and may be sensitive to the achieved quality of the concrete. Until equations can be modified to allow for effects such as cover quality and cracking, they remain of most value in laboratory and test site studies where these can largely be controlled. The development of deterioration models for real structural members becomes even more complex when, for example, interactions between chloride presence and carbonation rates must be taken into consideration (22). Similar problems arise in the already mentioned case of sulphate attack by different salts or combinations of different salts.

In his review published in a recent symposium, Mehta (23) states that although significant advances have been made in regard to the understanding and control of various physical and chemical phenomena contributing to concrete deterioration, the trend towards less durable concrete structures has yet to be reversed. In supporting his view, he points to current information from tests on durability being in a fragmentary form resulting in an over-reliance on test methods and specifications. He goes on to suggest that accelerated laboratory tests do not correlate well with the behaviour of concrete structures in practice.

If it is accepted that the necessary data for undertaking full risk analyses of durability problems of concrete structures is not yet available to designers, one is forced to consider what approach is now adopted to enhance long-term durability. At the outset, the as yet imperfect models of expected deterioration mechanisms are applied to structures or concrete types. The resulting estimations are then reinforced by using materials for which laboratory data indicate an expectation of improved durability. The necessity to develop and improve both the models and the materials is obvious, and both challenge the ingenuity of the concrete technologist interested in durable structures.

Classification of Concrete Deterioration

Five classes of concrete deterioration are recognised by ACI Committee 201. These are:-

1. Freezing and thawing,
2. Aggressive chemical exposure,
3. Abrasion,
4. Corrosion of steel and other materials embedded in concrete, and
5. Chemical reactions in concrete.

As pointed out by Popovics (24), such classification is based on factors that can cause deterioration. That author proposed a new classification based on the mechanisms of deterioration. Consideration of deterioration phenomena in this manner, although most helpful to an investigator, does little to further expand the practitioner's scope to limit potential problems.

Neville (25) suggests three factors to be mainly responsible for many of the current problems relating to the durability of concrete. First, he suggests that the acceptance of concrete on the basis of strength specifications alone is inadequate. Secondly, he feels that the use of leaner mixes to achieve specified strengths through changes in cement properties have contributed to durability loss. Reinforcement corrosion in concretes, particularly those in hot climates, is noted as a third major factor with respect to concrete durability.

In a recently published book, a somewhat different approach has been adopted by Campbell-Allen and Roper (26). They consider durability and serviceability problems may arise due to first, "as built concrete properties" such as excessive permeability and internal chemical disruption, and secondly, "external influences" such as for example climatic and weathering effects, external chemical attack, wear, erosion and last, but not least in their influence, design errors and construction problems. The interactions between the concrete properties and external influences may lead either to deterioration or durability of the structural member. This approach reflects, at least in part, Mather's opinion that every environmentally induced crack represents the consequences of an imperfect choice in matching the concrete selected for use in a given environment with the characteristics of that environment (27).

In this Chapter, certain of the most important features of durability are considered and a general attempt has been made to follow the ACI classification of concrete deterioration.

RECENT INFORMATION ON DETERIORATION PHENOMENA

Much has been written on the various phenomena which may cause premature deterioration of concrete in service. A general review of recent information relating to basic and applied research aspects and in service observations is presented in this Section. Much of the information is taken from ACI journals and from a series of Symposia sponsored by that body (28,29,30,31,32,33). Deterioration phenomena, as investigated in this chapter, are summarised in Fig. 1, based on the classifications adopted by ACI Committee 201. The performance of concrete using recently introduced materials is included as a separate segment. Specific topics relating to concrete durability considered under the classification of Aggressive Chemical Exposure are summarized in Fig. 2. Similarly, those topics relating to durability considered under the heading of Corrosion of Steel and Other Materials Embedded in Concrete are summarized in Fig. 3, whereas those relating to Chemical Reactions in Concrete are summarised in Fig. 4. It must be noted that topics within these groups are by no means exhaustive. Nevertheless, they serve as a basis for considering the advances made within the field of work on concrete durability in recent times.

Freezing and Thawing

Durability loss through freezing and thawing action is common in concrete structural elements in many parts of the world. The most common procedure to overcome this problem is the introduction into the concrete of a given proportion of air bubbles having a specific distribution throughout the paste. Air entraining admixtures are generally anionic surfactants which, due to their adsorbed orientation, form stabilised bubbles in the paste.

Mather (34) states that concrete will be immune to the effects of freezing and thawing if the pores containing freezable water are never more than 91% filled ie. not critically saturated or if they are, the paste has an air-void system with an air bubble located not more than 0.2 mm from such a pore. He also points out the importance of sound aggregate and moderate maturity of the paste. Despite such available information, problems still arise. Such problems are often the result of the introduction of new materials into the technology. Silica fume and superplasticisers have imparted different properties to concrete mixtures, and their interactions with traditional air entraining admixtures are still the focus of intense investigation.

As mentioned above, critical to the durability of concrete undergoing freezing and thawing mechanical actions is the air-void spacing factor (ASTM C 666). Concerns have been expressed with regard to the influence of silica fume replacements of cement in concrete and resulting freezing and thawing durability. Pigeon, Aitcin and Laplante (35) concluded that it was possible not only to produce field silica fume concrete with a satisfactory air void spacing factor but to do so without affecting the stability of the air void system. In further studies by Pigeon, Plante and co-workers (36,37,38), it was concluded that silica fume and type of mixing have little influence on the production and stability of the air-void system but that superplasticizers could cause a significant increase of the spacing factor value. The influence of superplasticisers on the air-void stability was found to be, in turn, influenced by the cement type and the air entraining admixture. Difficulties were encountered in estimating correctly the value of the spacing factor from the air content of fresh concrete.

It is important to be able to modify mixtures when retempering is carried out or when tests indicate unsatisfactory entrainment. Studies on retempering carried out by Pigeon, Saucier and Plante (39) have demonstrated that it is possible to entrain an adequate air-void system with a spacing factor of less than 200 microns if a normal dosage of the air-entraining agent, with a little water, is incorporated into the mix 45 minutes after the initial water-cement contact. An unsatisfactory air-void system can be corrected by adding more of the air entraining agent after 45 minutes, but the quantity added must be great enough to lower significantly the value of the spacing factor. The results further show that air-content variations lower than 2% should not be used to predict spacing factor variations.

Freezing and thawing mechanical actions when coupled with the ingress of deleterious salts can reduce significantly the longevity of structures. Moukwa (40) reports that wetting and drying of the concrete together with freezing and thawing thermal shock actions generate microcracks thus allowing deep penetration of aggressive ions. Such ions were found to react with the cement paste and cause further disruption, but the addition of either blast furnace slag and silica fume to concrete may reduce such disruption as they densify the matrix and improve the paste-aggregate bond. Silica fume additions were found by Moukwa to reduce the disruption to a greater degree than blast furnace slag.

A large scale project relating to the field performance of a series of different concrete types has been one focus of work for CANMET and collaborating organisations. Malhotra, Carette and Bremner (41) have recently reported on this work. Normal and lightweight concretes incorporating supplementary cementing materials in a marine environment (at Treat Island, Maine, USA) are being evaluated in long-term field exposure tests. The authors have reported that after up to 9 years' exposure, both normal-weight and lightweight air-entrained concretes show no degradation of the mass of the concrete. Some of the specimens have, however, shown significant surface deterioration. The amount of deterioration was generally found to increase with an increasing water:cementitious ratio and with increasing replacement of cement with slag and fly ash. It appears that surface deterioration can be avoided if the portland cement content is kept above a certain minimum level. The researchers have found that over a long exposure duration, non air-entrained concrete is not durable in this environment.

The methods of avoidance of disruption due to freezing and thawing phenomena are well documented as are the mechanisms thereof. At the present time interest is centred on the bubble size and distribution requirements of special mixtures. Simply because most attention has been paid to paste properties the potential detrimental influence of certain porous aggregates should not be ignored.

Aggressive Chemical Exposure

Sulphate Attack--Cohen and Mather (1) suggest that the mechanism of sulphate attack on portland cement concrete is not yet well understood. They suggest that this factor has limited the confidence that can be placed in and hence the reliability of existing standard tests and models to predict performance and service life of concrete subjected to sulphate attack. They suggest a need for both a better understanding of the sulphate attack mechanism and the development of more reliable standard tests and better models to predict performance and service life (1). An approach for assessing the influence of pozzolans, or other cementing materials, on concrete performance and service life in sulphate environment is presented.

Rasheeduzzafar and co-workers have recently re-investigated the influence of cement composition on the sulphate resistance of concrete (42). Acquired sulphate deterioration data indicated that, in addition to the C_3A content, the C_3S to C_2S ratio of the cement had a significant influence on sulphate resistance. They cite an example where comparisons were drawn between the sulphate deterioration for Type I portland cements having C_3A levels of 11.9% and 9.3%. It was reported that the C_3S to C_2S ratio for the cement having the higher C_3A was 7.88 whereas the C_3S to C_2S ratio of the cement having the lower C_3A was 2.57. After 150 days of exposure to an accelerated sulphate test, the Type 1 cement having the higher C_3A level was found to have a level of deterioration that was 2.5 times higher than that for the Type I portland cement having the lower C_3A level. Significant retrogression of strength was observed on immersion in sulphate solution even for a Type V (C_3A = 1.88%) sulphate-resisting cement that had the high C_3S to C_2S ratio of 5.28. On the other hand, their accelerated sulfate-resistance tests show that a binder composed of 20% silica fume blended with a Type I cement having a 14.00% C_3A content performed 1.4 times better than a Type V portland cement having 1.88% C_3A.

Moukwa (43) investigated attack of cement paste by $MgSO_4$ and $MgCl_2$ solutions. Mercury intrusion porosimetry was used to analyse the paste pore structure and distinctive characteristics of each type of corrosion were found. A densification of the pore structure was observed with $MgSO_4$ whereas $MgCl_2$ corrosion was associated with an opening of the pore structure. Mineral additives were found to protect 14 day cured cement paste samples against the $MgSO_4$ attack. However, a longer initial curing period was found to be necessary to mitigate $MgCl_2$ attack.

The effects of magnesium sulphate and sodium sulphate solutions on the durability of 0.30 water-to-solid ratio pastes of ASTM Types I and V portland cements with partial replacement of 15 mass percent silica fume have been investigated by Cohen and Bentur (44). Results show that silica fume addition to portland cement can improve resistance to sodium sulphate attack, but it can greatly impair resistance to magnesium sulphate attack . The greater damage effected by the magnesium sulphate solution on portland cement and portland cement plus silica fume specimens is due to the decomposition of the C-S-H gel to M-S-H (magnesium silicate hydrate) which is non-cementitious. Furthermore, the authors conclude that the greater intensity of the attack on portland cement plus silica fume specimens as compared to the portland cement specimens may be due to the absence of magnesium hydroxide in the portland cement plus silica fume paste. The authors concluded that there was a pressing need to establish failure criteria and performance classifications for sulphate resistance in the light of their studies. These criteria were thought to be dependent primarily upon mixture proportions, curing, duration, type of sulphate exposure, and indicator of deterioration (i.e., changes in length, mass, compressive-flexural strength, and modulus of elasticity).

The straightforward relationships which earlier governed cement selection for sulphate environments must now be questioned when mineral additives are to be used. More details are provided by Berry and Malhotra (45) for fly ash, Sellevold and Nilsen (46) for silica fume and Douglas (47) for slags.

Sea Water Attack--Using information from recently reported case histories of concrete deterioration in sea-water, Mehta presented a summary on this subject at the CANMET/ACI International Conference on the Performance of Concrete in Marine Environment (48). He confirmed that harmful chemical reactions between sea-water and the constituents of hydrated cement paste which lead to carbonation, sulphate attack, and magnesium ion attack can be limited to the surface when measures to ensure low permeability of concrete are successfully applied. He considers the ACI and FIP Recommended Practice for Off-Shore Concrete Structures, and provides comparisons with field experiences from North Sea structures. Since high-strength concretes (50 to 70 MPa), containing water-reducing and mineral admixtures are relatively impermeable, in the opinion of that author they offer an excellent solution to the problem of durability of concrete in a sea-water environment.

Organic and Inorganic Acid Attack--The attack on concrete by acids takes many forms. The acids to which concrete is exposed may be very dilute such as those in food preparation areas and acid rain, to quite concentrated such as in sewers (49).

Kong and Orbison (50) have conducted studies into concrete deterioration due to acid precipitation. An accelerated laboratory test program demonstrated the response of portland cement concrete cylinders to simulated acid precipitation solutions. Concretes having 28 day compressive strengths between 21 MPa and 62 MPa were subjected to test. The results indicated that specimen deterioration, as evidenced by material loss and discolouration, increased as the solution pH decreased. Specimen deterioration, surprisingly, also increased with increasing concrete design compressive strength.

Fatthui and Hughes (51) conducted studies on cement paste, mortar, and concrete mixtures containing ordinary portland cement suspended in a channel containing sulphuric acid solutions. Investigated were the effects of cement content, water:cement ratio, fly ash addition, polyvinyl alcohol inclusion and the use of a high-range water-reducing admixture. Specimen weight change data indicated that the rate of deterioration of cubes increased with increases in cement content, water:cement ratio and acid concentration. Additions of polyvinyl alcohol appeared to enhance the resistance and integrity of the cubes when in contact with the acidic solutions. The same authors conducted further studies into the performance of cement paste and concrete subjected to sulphuric acid attack (52). The results of these studies again indicated that the deterioration of the cubes for a two percent sulphuric acid concentration increased with an increased cement content.

Attiogbe and Rizalla (53) also conducted studies on the sulphuric acid attack on concrete. The response of four different concrete mixtures to sulphuric acid attack were evaluated in an accelerated laboratory test program. Increases in thickness (expansion) of small specimens (with large surface area-to-volume ratios) reflected more consistently the effects of sulphuric acid concentration on concrete when compared with measurements of the weight loss of larger specimens. Photomicrographs of the concrete microstructure showed that concrete deterioration started from the acid-exposed surface and progressed inward. The degree of concrete deterioration increased under alternate wet-dry cycles of exposure using sulphuric acid.

The problems of acid attack on cement products do remain; they can, in part, be mitigated by selection of particular materials. Nevertheless, the use of coatings or polymer impregnation methods would seem to present the most helpful solutions under all except the most mild acidic exposure conditions, when the use of carbonate aggregates may provide satisfactory service.

Abrasion

Although the ACI has a category specifically designated Abrasion, erosion and cavitation are best discussed here-under. Certain structural and architectural concrete surfaces are subject to abrasive, erosive and cavitational conditions over at least some of their service lives. Although much work has been published on the resistance to abrasion of different concretes in certain environments (54,55), it is difficult to fully cover this subject in detail as abrasion can be imparted to concrete surfaces in a multitude of forms. Trafficked surfaces, structural elements in contact with deicing salts and hydraulic structural elements would each represent unique potentially abrasive conditions. The most appropriate concrete type for each application would depend on the prevailing conditions.

Major causes, controls, maintenance policies, and repair of erosion in hydraulic structures were discussed in a report published by ACI Committee 210 (56). They reported erosion to occur as a result of three major causes, (i) cavitation, (ii) abrasion and (iii) chemical attack. Design parameters, materials selection and quality, environmental factors, and other issues affecting the performance of concrete were discussed. For given operating characteristics and conditions to which a hydraulic structure would be subjected, the Committee was of the view that it was possible in the design of these structure types to mitigate future erosion of the concrete. Most of the failures in these types of structure were attributable to either changes in operational factors or incorrect assessments of design service conditions which resulted in erosion of concrete surfaces and subsequent requirement for repair.

Corrosion of Steel and Other Materials Embedded in Concrete

The reader interested in the corrosion of steel and other embedded materials is referred to work of ACI Committee 222 (57), and the many recent publications on the subject (58,59). A few recent advances have been selected for consideration here.

Studies on Pore and Synthetic Solutions--A variety of data have been published on the corrosion characteristics of steel in expressed paste and mortar pore solutions as well as synthetic solutions approximating their compositions. Although much useful information can be acquired on the steel solution system through such studies, their relevance and application to real concretes is yet to be established.

Cao, Baweja and Roper (60) used potentiodynamic anodic polarization procedures to model the corrosion characteristics of steel in solutions derived from cements and blended cements and found differences in the performance of binders tested. Solutions for this exercise were prepared by mixing cement with water at a water:binder ratio of 2.0 and seperating them from the solid phase after 3 and 7 days. It was found that solutions made from cements high in C_3A and from slag blended cements displayed better steel passivation characteristics than those derived from cements which were low in C_3A. Subsequent work on concretes made with these particular cements has confirmed this trend.

Laboratory studies of the composition of pore solutions expressed from hydrated portland cement pastes containing chloride and sulphate salts have been conducted by Treadaway, Page, and Macmillan (61). The authors concluded that chloride-induced corrosion of steel in concrete will tend to be promoted by the use of cements low in C_3A and by the presence of sulphate salts as mixture contaminants. They verified the above predictions in field exposure trials using reinforced concrete specimens.

Goni and Andrade (62) studied synthetic concrete pore solutions having chlorides included at a range of concentrations. Chlorides were included in the form of NaCl or $CaCl_2$ and solution pH ranged between 11.64 to 13.22. In their studies, relationships between bulk chemical parameters and mean corrosion intensity were established. Corrosion intensity was measured using polarization resistance procedures.

Derrien and co-workers (63) conducted an experimental investigation into the electrochemical behaviour of steel embedded in concrete using cathodic polarization procedures. Solutions, simulating concretes of various ages having undergone different degrees of carbonation, were studied. These solutions contained a range of chloride ion concentrations. Higher chloride ion concentrations promoted steel depassivation and, under certain conditions when corrosion occurred, a white deposit, described by the authors as a mineral scale, appeared simultaneously with brown oxide product at different locations on the steel. Roper (64) has described the presence of blebs of chloride rich material at scale-metal interfaces from corroding reinforcement.

Influence of Permeability and Paste Structure--Two very important parameters which influence the corrosion characteristics of steel in concrete (in the absence of any cracking) are the permeability and the paste structure of the concrete. Numerous investigations have been conducted in this field.

Detwiler, Kjellsen, and Gjorv (65) investigated the resistance of chloride intrusion into concrete cured at different temperatures. The authors point out that first, higher curing temperatures result in a coarser pore structure and secondly, elevated curing temperatures reduce the long-term strength of concrete. They reported results of work on a range of concrete types which showed that at a given water:cement ratio, elevated curing temperatures increased chloride diffusion into portland cement concretes, and consequently led to early depassivation of reinforcement. This effect was found to be more pronounced at lower water:cement ratios than at higher ones. They concluded that these findings need to be taken into account in the construction of concrete structures for which durability is a concern.

Banthia and Mindess (66) reported on work which investigated the permeability of cement paste to water using a triaxial permeability cell. A new technique of specimen conditioning based on cyclic flow reversal was devised by the authors. It was used for the early attainment of equilibrium flow conditions. Effects of cement type, age, and silica fume addition were investigated. The authors found the coefficient of permeability to decrease with an increase in the degree of hydration. The use of silica fume was found to decrease the permeability, moreover, permeability varied with the level of silica fume addition.

Influence of Carbonation--Much work has been published on the influence of carbonation on concrete durability, corrosion of reinforcement and to the performance of in situ concrete structural elements. Recent work by Richardson (18) has considered this topic in detail. Although excessive carbonation in concrete is undesirable, its precise influence on the longevity of structural members in service is still debated.

Ho and Lewis (67) have published widely on the topic of carbonation and its prediction. Their studies have included concretes incorporating various constituents including chemical admixtures and fly ash. They concluded that for concretes which had limited initial curing, the water:cement ratio, and not the water:binder ratio, was the most reliable parameter for predicting the resistance of concretes to carbonation.

Kobayashi and Uno (68) conducted accelerated carbonation tests on mortar specimens made using a range of cement types. It was found that the rate of carbonation increased with greater alkali contents in the cement. The authors demonstrated this trend to become even more important when diffusion of the pore solution occurred.

Hobbs (69) conducted an investigation into the influence of 35% replacement of one normal portland cement by two fly ashes. The investigation focused on the rate and depth of carbonation, strength development and water retention of a set of concrete cubes. He concluded that both water retention and strength development characteristics of fly ash concretes may be more adversely affected by short-term curing than those of normal portland cement concretes. He further concluded that the depth of carbonation in concretes of a similar water:binder ratio or similar binder content could be higher in fly ash concretes than in normal portland cement concretes. Although such findings have been reported in other laboratory programs where accelerated carbonation tests have been carried out, field studies on fly ash and other concrete types have not led to similar conclusions (70).

Bier, Kropp, and Hilsdorf (71) investigated the carbonation characteristics of paste, mortar and concrete specimens made from different cement types. A range of water:cement ratios were used to cast specimens. One set of specimens was carbonated under normal conditions and a further set under accelerated conditions (2% CO_2 by volume). The authors concluded that the alkaline environment in the paste could be restored after carbonation by applying a mortar layer to the carbonated surfaces. They suggested that it would be thus possible to re-establish passivation for embedded reinforcement and that the rate of realkalinization was dependent upon the phase composition and the pore structure of the carbonated samples.

Influence of Chloride Ions--Chloride ions, abundant in sea water, are known to depassivate steel embedded in concrete even where the alkalinity of the pore solution is high. Three suggested mechanisms for chloride induced corrosion have been described by ACI Committee 222 (Corrosion of Metals in Concrete). Extensive studies into chlorides and concrete durability have been conducted and readers are again referred to the publications cited at the start of this section.

Hansen (72) notes that a well known relationship between water:cement ratio and water permeability was developed many years ago by Powers and co-workers for cement pastes under standard laboratory conditions. In codes and standards this relationship has erroneously been applied to concrete which is inadequately cured at high temperatures and lower humidities in hot and dry tropical countries. It is suggested that this may explain why so many marine structures are failing due to reinforcement corrosion in hot dry climates.

Moukwa (73) investigated the penetration of chloride ions into mortar specimens exposed to sea water at 20°C, -1°C and under freezing and thawing conditions. He found chloride ions to penetrate deeper into the mortar specimens at low temperatures. He concluded that the volume of macropores controlled the penetration. He further concluded that inclusions of silica fume, by densifying the matrix, blocked this penetration but had a tendency of increasing the chloride ion content within the surface layer.

Maslehuddin and co-workers (74) reported results of experiments carried out to evaluate the compressive strength development and corrosion resisting characteristics of concrete mixtures in which fly ash was used as a sand replacement for cement. Concrete samples were partially immersed in a 5% sodium chloride solution for a period of about 4 years. Results showed that additions of fly ash as an admixture increased the early age compressive strengths and the long-term corrosion resisting characteristics of concrete. The authors concluded that the superior performance of the fly ash concretes was attributable to the densification of the paste structure due to pozzolanic reaction.

In another investigation, Maslehuddin and co-workers (75) investigated the long-term corrosion resisting characteristics of concretes made with two fly ashes, a natural pozzolan, a blast-furnace slag cement, and a preblended portland-fly ash cement. Specimens were again immersed in a 5% sodium chloride solution for a period of more than five years. Results indicated that additions of pozzolans and slag increased the corrosion resistance of concrete. The corrosion rates of steel in concretes made with the blast furnace slag cement were found to be lower than those for any of the other concretes.

Jaegermann (76) reported on an investigation into the influence of water:cement ratio and curing on chloride penetration into concrete specimens. He concluded that there was a linear relationship between observed chloride threshold levels and water:cement ratio. A linear relationship between chloride ion concentration and curing duration was found for exposure data less than one year. This relationship was found not to hold for longer exposures.

Jensen and Pratt (77) studied the hydration and microstructural development of Portland cement and fly ash blends in the presence of chlorides. It was found that the initial rate of hydration and pozzolanic reaction were accelerated when the mixing water contained chlorides. Chlorides were found to complex as chloroaluminate or chloroaluminoferrite hydrates, which formed from reacting C_3A, ferrite and calcium aluminate hydrates present in the cement pastes. The authors suggested that additional chlorides may be complexed in fly ash cement blends from calcium aluminate hydrates produced by the pozzolanic reaction.

Byfors (78) carried out investigations into the chloride diffusion characteristics of paste samples made from silica fume-cement or fly ash-cement blends. He found that silica fume and fly ash inclusions considerably reduced the chloride diffusion rate. He suggested other factors of major importance for the resistance of concrete to chlorides to be first, the composition of the pore solution and secondly, the capacity of the cement paste to bind chloride chemically. Through conducting measurements of pH in expressed pore solutions, he concluded that although mineral additions reduce the pH, and hence the capacity of the cement paste to resist reinforcement corrosion, the addition of silica fume or fly ash in appropriate amounts could extend the initiation time for diffusion controlled and chloride initiated reinforcement corrosion.

Influence of Oxygen and Water Diffusion--The generally accepted view of the corrosion of steel embedded in concrete assumes that the reduction of oxygen supports the overall process of corrosion in this medium. Gonzalez and co-workers (79) showed in their studies that, under certain circumstances, the corrosion rate of steel in concrete could be high even though the supply of oxygen is small or negligible. They went on to suggest that the corrosion mechanism appeared to require cathodic processes which did not depend upon oxygen diffusion. They suggested that crevices at the steel to concrete interface coupled with relatively low pH environments presented a possible explanation for the observed high corrosion rates of steel in concrete in the absence of significant amounts of oxygen.

Influence of Sulphate Ions--Sulphate ions are also known to cause corrosion of steel in concrete and some work has been done on the mechanism of breakdown in passivity and subsequent durability loss.

Cheng, Lee and Tsai (80) presented findings of their laboratory investigation in which the roles of chloride ions and sulphate ions on reinforcing steel corrosion were studied . Low carbon steel bars were embedded in the concrete specimens made with either a Type I or a Type V portland cement. They concluded that the effects of chloride ions and sulphate ions on the electrochemical properties and corrosion mechanisms of reinforcing steel in concrete were quite different, and that polymer impregnation would increase the resistance of concrete to the deleterious agents.

Al-Tayyib and co-workers (81) conducted a laboratory investigation into the influence of sulphate ions on reinforcing steel corrosion. Tests were carried out at 22°C and 50°C. Their results indicated that, in the presence of sulphate ions at 22°C, active corrosion resulted from the modification of the protective passive film to a sulphate film which was found to be less protective than that of the original iron oxide film. They concluded that the combined effect of the presence of sulphate ions and the elevated temperature resulted in a seven fold increase in the observed corrosion rate when compared with that of chloride induced corrosion at a normal temperature.

Chemical Reactions in Concrete

Magnesia Unsoundness--The testing of soundness of cement so as to ensure serious expansion does not occur, has been so perfected that problems due to the late hydration of periclase to brucite are very rare. The same is true for disruptive expansion due to residual calcium oxide in cement clinker. Nevertheless, for new sources of cement, such reactions must be considered as they may prove costly if not avoided.

Non-Optimum Sulphate--The advantages of producing portland cements having optimum sulphate contents are maximum strengths, minimum shrinkage and creep. Problems arise because the optimum quantity of gypsum required is influenced by temperature of mixing. Under low temperature conditions, cements at optimum for 20°C may expand excessively. Such expansion may lead to durability problems. More importantly, much more work in future needs to be directed to the establishment of optimum sulphate contents of blended cements. According to Cook, Hinczak and Cao (82), the hydration reactions of slag activated by portland cement, gypsum and alkali are probably different for each activator. In general, however, the literature contains rather little information in this area. The use of ground limestone in portland cements will stimulate work on the sulpho-aluminate, carbo-aluminate and chloro-aluminate hydrate phases. It is known that the solid solution series of these hydrates may be major contributors to durable or non-durable concretes.

Alkali-Aggregate Reactions--Work on this subject has been focused on two main areas. First has been the refinement of test methods which predict the performance of cement-aggregate systems with regard to potential alkali-aggregate expansion. The second has been studies of field concretes in which alkali-aggregate reaction has taken place.

Hudec and Larbi (83) have presented results of accelerated tests for screening potentially reactive concrete aggregates. The tests were performed using computer-controlled instrumentation. Their statistical analysis indicated highly significant correlations between the accelerated test and the standard tests (ASTM C 227-81 (84) and CSA A23.2 14A (85)) adjudged in the study. From the relationships between the accelerated test and the standard tests, critical limits of expansion were established for the former. When tested in hot (80°C) 1N NaOH solution expansions greater than 0.17% after 12 days and 0.33% after 24 days were considered excessive for alkali-silica reactive rocks. For alkali-carbonate reactive rocks, the critical limit was found to be 0.16% after 24 days. Larbi and Hudec (86) in another study considered the accelerating effects of hot and cold saturated sodium chloride solution on concrete containing alkali reactive aggregates. Their results showed that hot saturated NaCl solutions accelerated alkali reactivity and the results correlated well with the ASTM C 227-81 and CSA A23.2 14A. They found, conversely, that in cold saturated NaCl solutions, the alkali reactivity was rock-specific and slow. The test, when conducted at low temperatures was judged unsuitable for screening alkali-silica reactive rocks. The authors also concluded that although alkali ions introduced from deicing salts do initiate alkali-carbonate reactions, the reaction rates were slow under ambient temperature conditions.

The influence of calcium chloride based accelerators an ASR has been studied by Wang and Gillott (87). They concluded that mortar bars containing 2% opal in the presence of the accelerator showed higher expansions than bars made without the accelerator. They also noted a decrease in expansion when silica fume was used as a partial replacement for cement.

Recently, a detailed study of the effects of alkali-aggregate reaction on the engineering properties of concrete including compressive and tensile strength, elastic modulus, and pulse velocity has been reported by Swamy and Al-Asali (88). Two types of reactive aggregate - a naturally occurring Beltane opal and a synthetic fused silica - were used. Tests were carried out at 20°C and 96% relative humidity. The results showed that losses in engineering properties do not all occur at the same rate nor in proportion to the expansion undergone by the ASR-affected concrete. The two major properties affected by ASR were flexural strength and dynamic modulus of elasticity. Change in compressive strength was not a good indicator of ASR, but the flexural strength proved to be a reliable and sensitive test for its monitoring. Non destructive tests such as dynamic modulus calculations and pulse velocity measurements were also able to identify deterioration of concrete by ASR. The data indicate that acceptable expansion limits due to ASR depend on the type and use of a concrete structural member.

Test data on the use of a low-calcium Class F fly ash to control the structural deformations occurring in singly reinforced concrete beams undergoing ASR have been reported by Swamy and Al-Asali (89). A highly reactive aggregate which produced concrete having an expansive strain of about 1.5% and a moderately reactive aggregate causing a maximum concrete expansion of about 0.6% were used. The behaviours of reinforced concrete beams containing 0, 30%, and 50% cement replacement by weight of fly ash were monitored over a period of 2 years to study the controlling influence of the pozzolan on strains, cracking, and deflection. The beams were subsequently loaded to failure to assess the effect of fly ash on the stiffness, strength, and failure modes of the ASR-affected beams. The authors concluded that fly ash had a dual role in controlling deformations and reducing strength loss due to expansion.

Durand et Al (90) found that the use of silica fume reduced the amount of expansion due to alkali-aggregate reactions on tests carried out on mortar bars. Parallel studies on expressed pore solutions from cement paste showed that cement replacements with silica fume reduced the Na^+, K^+ and OH^- concentrations within the solutions. The relationship between the change in pore solution composition with the silica fume pastes and expansions was better demonstrated by the accelerated mortar bar test than by the standard concrete prism test. This was also true when the reductions of expansion for the two sets of results were plotted as a function of the silica fume content.

Data on concrete expansive strains due to alkali-aggregate reaction were reported for four high strength concrete bridge structures in Queensland, Australia by Carse and Dux (91). The bridge structures were composed of prestressed beam elements supporting a concrete deck. Alkali-aggregate reaction was observed almost exclusively in steam cured prestressed concrete beams. Two of the structures were formed in part by prestressed concrete beam units which were found to be expanding at significant rates after 10 years service. The other two bridges were formed of prestressed beam units. Only in localised areas were members found to be expanding. Members which were found to be experiencing significant general expansion were made with reactive crushed river gravel aggregates. Members undergoing local expansions contained crushed quarried rhyodacitic tuff aggregates. Reasonable agreement was observed between crack widths and the measured strain increase on the soffits of prestressed beams.

Although a considerable amount of information has been published on alkali-aggregate reaction, problems still arise. The low degree of certainty with which problem materials can be detected still leaves a considerable amount of work to be done. Other areas now attracting attention of workers are the repair problems associated with structural members already affected by ASR, and the wider use of pozzolans and coatings to avoid the problem.

PERFORMANCE OF CONCRETE USING RECENTLY INTRODUCED MATERIALS

Fairly recent introductions of admixtures, additives and coatings have significantly influenced durability expectations. Amongst these are superplasticisers and calcium nitrite admixtures, silica fume, and silane coatings. Certain of these have already been considered.

The influences of superplasticisers on durability and air-void systems have been covered by Malhotra (92) in a review paper at the Third International Conference on Superplasticisers and Other Chemical Admixtures in Concrete. The conclusion is drawn that significant benefits can be derived from its use.

Alonso and Andrade (93) conducted studies into the effect of sodium nitrite as a corrosion inhibitor. Polarization resistance and electrochemical impedance measurements were the techniques used to track the corrosion process. Depending on the type of cement and the sodium nitrite proportion employed, it was found that the attack was reduced and even completely avoided when carbonation was the sole aggressor. When chlorides and carbonation acted together, however, the proportions of sodium nitrite used (2% and 3%), were not enough to reduce the attack, and the reinforcing steel corroded at a rate similar to that when no nitrites were present in the mix. El-Jazairi and Berke (94) are more enthusiastic about the capability of calcium nitrite as a corrosion inhibitor in concretes exposed to chloride ions and they present relationships between the amount of calcium nitrite required for protection at different chloride levels. A paper in this volume by Berke and his associates discusses in depth the role of calcium nitrite to control corrosion.

With the introduction of silica fume and superplasticisers to form high strength concretes, attention has been drawn by Mehta (95) to the importance of microcracking in promoting deterioration. Such microcracking may be caused by the use of unsatisfactory aggregate materials in high strength mixtures. A detailed document on condensed silica fume in concrete including durability effects has been prepared by FIP (96).

Much interest has recently been aroused in concrete coatings for improved durability. Consider a guide for the design and construction of concrete parking lots prepared and reported by ACI Committee 330 (97). In considering maintenance and repair aspects, the Committee suggests that silane-siloxane coating materials are suitable water repellants for protecting concrete. These repellants have the advantage of allowing the substrate to breath normally, thus preventing damage from a build-up of moisture between the concrete and a film-forming material. They have also been proven effective as chloride screens, thus protecting the concrete from deicing chemicals in cold climatic conditions and airborne salt in marine and coastal areas. The use of silanes for the protection of bridge decks is increasing in both North America and Europe. It is probably too soon to adjudge their effectiveness in overcoming long-term durability problems under service conditions, however, test site and laboratory studies suggest that they will provide a significant advance in overcoming some problems of non-durable concrete.

CONCLUDING REMARKS

The mechanisms of most durability problems are sufficiently well understood to allow avoidance of most of them. In certain cases, choices of materials or techniques known to improve durability may clash with the preconceived ideas on design and construction procedures employed by engineers. It is important therefore that the information available to cement and concrete technologists be speedily shared with design engineers, and so enter practice. More durability problems arise than need to occur. If the application of our present knowledge of factors which influence durability were immediately and widely implemented, the degradation of infrastructure would be considerably lessened. How this is to be achieved is as much a process of teaching as deeper research into the fascinating fields of cement physics and chemistry.

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Table 1(a) - List of Deterioration Effects (After Roper, Kirkby and Baweja, Reference 10)

Deterioration Effects Numbers 1 to 30

Deterioration Effect		Description of Effect
Classifications	Code No.	
Metallic Corrosion	1	Metallic Corrosion - Rebar - Uniform
	2	Metallic Corrosion - Rebar - Localised
	3	Metallic Corrosion - Prestressing - Uniform
	4	Metallic Corrosion - Prestressing - Stress Corrosion Cracking
	5	Metallic Corrosion - Prestressing - Pitting
	6	Metallic Corrosion - Embedments - Rod - Uniform
	7	Metallic Corrosion - Embedments - Rod - Localised
	8	Metallic Corrosion - Embedments - Plate - Uniform
	9	Metallic Corrosion - Embedments - Plate - Localised
Popouts	10	Popouts
Spalling	11	Joint Spalls
	12	Edge Spalls
	13	Corner Spalls
	14	Surface Spalls
Cracking	15	Cracks Oriented wrt Rebar - Parallel - Colinear
	16	Cracks Oriented wrt Rebar - Parallel - Not Colinear
	17	Cracks Oriented wrt Rebar - Perpendicular
	18	Cracks Oriented wrt Columns - Circumferential
	19	Cracks Oriented wrt Columns - Radial
	20	Cracks Parallel and Colinear with Embedments
	21	Cracks Parallel and Not Colinear with Embedments
	22	Cracks Perpendicular to Embedments
	23	Cracks Within Embedment Projected Area
	24	Cracks External to Embedment Projected Area - Circumfrential
	25	Cracks External to Embedment Projected Area - Radial
	26	Cracks External to Embedment Projected Area - Parallel to Plate Sides
	27	Diagonal Cracks
	28	Random Individual Cracks
	29	Map Cracking
	30	Crazing

Table 1(a) - List of Deterioration Effects (After Roper, Kirkby and Baweja, Ref XX)
Deterioration Effects Numbers 31 to 50

Deterioration Effect		Description of Effect
Classifications	Code No.	
Scaling	31	Scaling and Peeling
Staining and Formations	32	Exudations
	33	Incrustations
	34	Efflorscence
	35	Discolouration
	36	Sand Streaks
Dusting	37	Dusting
Pitting	38	Pitting
Erosion	39	Erosion and Abrasion
Distortion	40	Deformation - Single Curvature
	41	Deformation - Double Curvature
	42	Warping
General Defects	43	Honeycombing
	44	Segregation
	45	Stratification
	46	Sand Pockets
	47	Water Pockets
	48	Drummy Areas
	49	Bug Holes
Delamination	50	Delamination

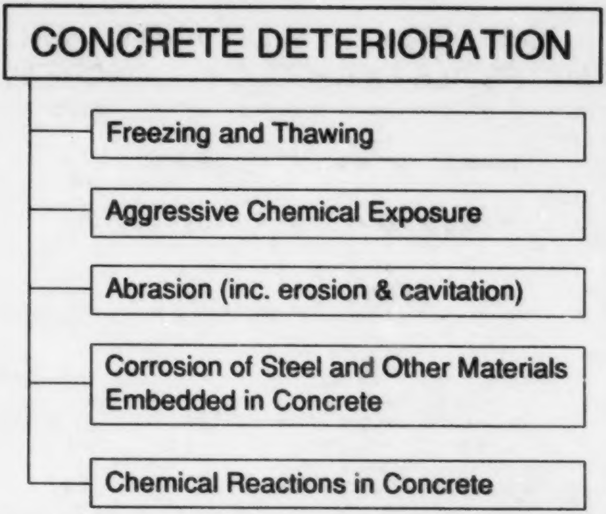
CONCRETE DETERIORATION
Freezing and Thawing
Aggressive Chemical Exposure
Abrasion (inc. erosion & cavitation)
Corrosion of Steel and Other Materials Embedded in Concrete
Chemical Reactions in Concrete

Fig. 1 - Broad Classification of Deterioration Phenomena Considered for this Study (After ACI Committee 201)

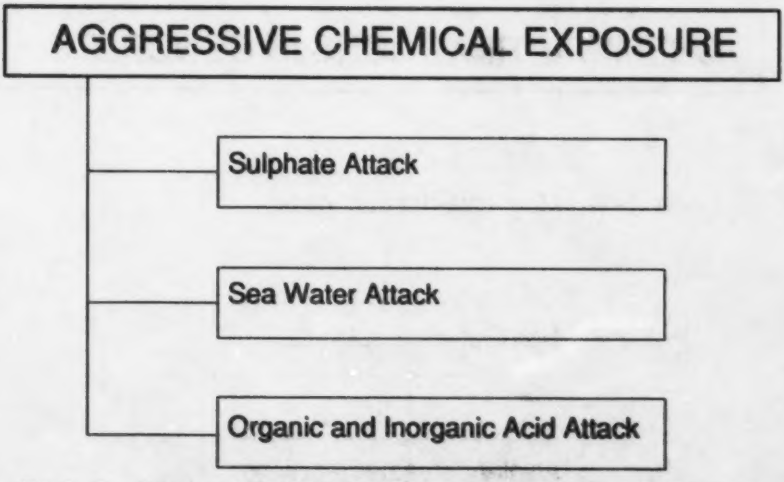
AGGRESSIVE CHEMICAL EXPOSURE
Sulphate Attack
Sea Water Attack
Organic and Inorganic Acid Attack

Fig. 2 - Consideration of Topics Under the Area of Aggressive Chemical Exposure

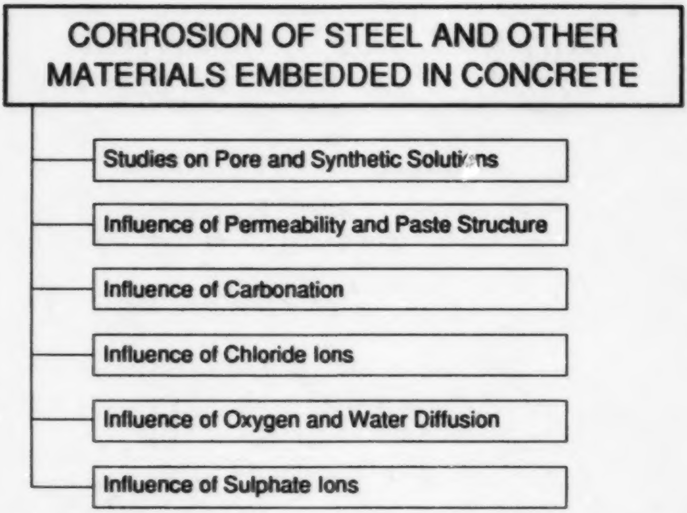
CORROSION OF STEEL AND OTHER MATERIALS EMBEDDED IN CONCRETE
Studies on Pore and Synthetic Solutions
Influence of Permeability and Paste Structure
Influence of Carbonation
Influence of Chloride Ions
Influence of Oxygen and Water Diffusion
Influence of Sulphate Ions

Fig. 3 - Consideration of Topics Under the Area of Corrosion of Steel and Other Materials Embedded in Concrete

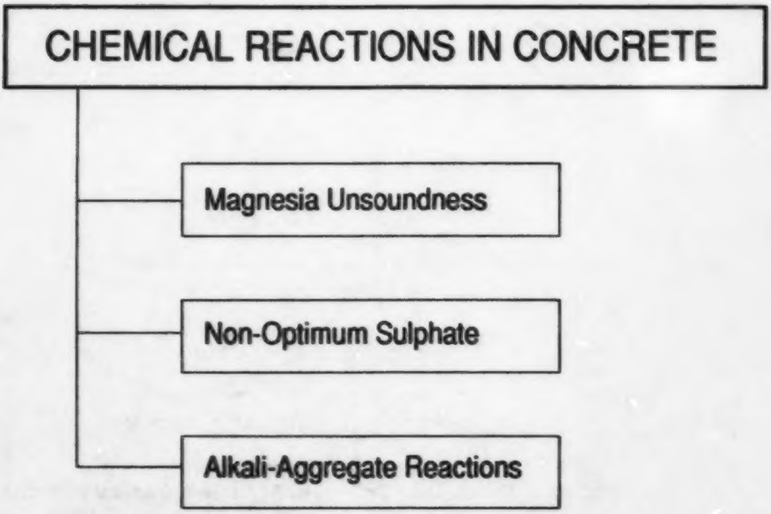
CHEMICAL REACTIONS IN CONCRETE
Magnesia Unsoundness
Non-Optimum Sulphate
Alkali-Aggregate Reactions

Fig. 4 - Consideration of Topics Under the Area of Chemical Reactions in Concrete

Alkali-Aggregate Reactions in Concrete:
Material and Structural Implications

By R. N. Swamy

Synopsis: This paper presents a critical evaluation of the material and structural implications of Alkali-Aggregate Reactions (AAR) in concrete. The mechanism of AAR, the types of reactive aggregates and the forms of reactivity are initially discussed. The factors influencing alkali-silica reactivity (ASR) are then presented in relation to performance of real structures. The effects of ASR on concrete expansion and on the properties of concrete are identified. The interrelationship between laboratory studies and field behaviour of ASR-affected structures is described. The structural deformations arising from ASR are discussed and the resulting effects on serviceability behaviour and ultimate strength of reinforced concrete beams, slabs and columns are analysed. Data are also presented to show how the cracking and disruption caused by expansive strains can be controlled through the use of mineral admixtures and reinforcement detailing. It is shown that structures can be designed to counteract the deleterious effects of ASR occurring at any time of their life.

Keywords: Alkali-silica reactions, concrete durability, expansion, strength, elastic modulus, structural deterioration, beams, columns, pozzolans, mineral admixtures, reinforcement, design.

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INTRODUCTION

Although concrete is a characteristically porous material, long experience and a good understanding of its microstructure and material properties have shown us that concrete is a reliable and sufficiently durable construction material. Concrete structures can also be built to give long, troube-free service life, as shown by many structures of Roman origin still seen in many parts of Europe today, provided care and control are exercised in its fabrication and production, and quality and workmanship are ensured in construction technology. Nevertheless, for a variety of reasons concrete as a material can deteriorate and concrete structures can show signs of distress in real life environments.

The sources which cause mateial deterioration and structural distress in concrete structures may broadly be classified into two. The damage may be due to either deficiencies in analysis, design, construction technology or quality of workmanship, or it may be caused by concrete and/or steel deterioration arising from a variety of phenomena, occurring alone or in combination, such as carbonation, penetration of chloride ions, chemical attack, freezing and thawing, and corrosion of steel. While forms of concrete deterioration due to external factors can generally be designed for and/or dealt with, material damage caused by internal chemical reactions arising from the binder material and the resulting structural deformations are much more difficult to tackle. When changes to internal microstructure are the primary cause of material and structural degradation, the effects on structural integrity can be aggravated by the environment. However, a combination of apparently unsuitable materials and unfavourable exposure conditions can be much more damaging than deficiencies in analysis or design, unless the engineering implications are anticipated and catered for at the design stage. High alumina cement is a typical example where the type of cement is the basic cause of concrete deterioration. With alkali-aggregate reactions (AAR), it is the cement matrix

- concrete aggregate reactions that are primarily responsible for the cracking and disruption of concrete (1) (Fig.1).

At first sight, and when looked at superficially, the problem of AAR may appear to be not unlike many of the other deteriorating processes that cause damage to concrete. Thus, like these other problems, AAR is a time-dependent phenomenon - it occurs during the service life of a structure, often after several decades of good performance, gives visible external warnings of internal damage, and generally appears to be harmless to the useful life of a structure. However, experience has shown, that although the actual number of cases of structures affected by AAR is few and far between, when considered in the context of concrete construction at a global level, the effects of AAR can be very profound and severely detrimental to a concrete structure affecting its strength, stiffness, serviceability, safety and stability (2, 3).

The phenomenon of AAR is very complex: there are many interacting and interdependent parameters that influence its occurrence, and even to date, there is probably no single test that can absolutely guarantee that a structure will never be affected by the disease. In several respects, therefore, AAR is an unusual and frightening phenomenon, not widespread or likely to be so, but nevertheless, quite devastating when it does occur. What makes this particular deteriorating process more unusual and complex than others is that AAR is much more difficult to recognize, identify and monitor. Further, it often provides a means for other deterioration mechanisms to develop and operate, and indeed, structures affected by environmental or structural deterioration other than AAR may also provide the required favourable conditions where a dormant alkali aggregate reactivity can be initiated and rekindled in the course of time. There is probably hardly any country which has not experienced it in some form or other, mild or intense; and the signs for fewer and fewer occurrence are not too hopeful when one considers the progressive depletion of our deposits of the so-called safe unreactive aggregates, the constraints of energy savings on the production and chemical nature of cement, and the continuing pollution of the environment. It might even be more realistic to say that perhaps and probably, there are no aggregates that are completely unreactive although they may remain, and appear to be so for a long, long time (4).

AAR is thus an unusual and unpredictable phenomenon. The factors involved in making up the reactivity are too complex: simplistic models belie the nature of the reactivity, and external agents, environmental conditions and microclimates can have as much a devastating influence as the basic ingredients of the reaction. The reaction can occur at a very early stage if all conditions necessary for such reactivity coalesce at that time. On the other hand, there is nothing to prevent the reactivity to lie dormant for many years or to continue at an unidentifiable and insignificant expansion level, only to be triggered off dramatically by simple changes in environmental conditions or failure of some other structural or construction component unconnected to ASR.

In spite of these uncertainties and unpredictabilities, careful examination and load testing of many AAR-affected concrete structures show that, whilst cracking can at times be visually frightening and severely damaging to the serviceability behaviour of the structure, the effects of AAR do not always undermine its overall functioning or strength (5-7). For example, the upper portions of the columns and the upper beam of a portal frame forming part of a major double deck road structure in Johannesburg in South Africa showed extensive visual deterioration and cracking. Comprehensive in situ load tests to 84% of the design load, and extensive measurements under load, however, concluded that whilst the AAR deterioration was alarming in appearance, it was not necessarily structurally dangerous (5). On the other hand, limited studies and load testing of the AAR-affected car park at Charles Cross in Plymouth, England, have shown that it was not possible to predict confidently the residual factor of safety in AAR-affected elements where detailing was critical (6).

MECHANISM OF ALKALI-AGGREGATE REACTION

In cracking, and the resulting deterioration arising from alkali-aggregate reaction in concrete is the direct result of chemical interaction between alkali hydroxides in the concrete, which are mostly derived from the cement, and reactive components in the aggregate particles used in the making of the concrete. There are thre known forms of alkali-aggregate reactivity, namely,

1) alkali-carbonate reactivity
2) alkali-silicate reactivity, and
3) alkali-silica reactivity.

Various types of reaction may occur in each class of these three reactivities, and not all of them are necessarily expansive or deleterious (4). Carbonate reactions may occur with calcitic limestones, dolomitic limestones or with fine-grained dolomitic limestone aggregates containing interstitial calcite and clay. The alkali-silicate reactions generally occur in alkali-rich concretes containing argillite and greywacke rock types in the aggregate: such reactions tend to be slow, and because such reactions may also occur with alkali-silica reactivity, complications often arise in identifying the reaction, and analysing results of diagnostic tests.

Of all the three types of reactivity, alkali-silica reaction (ASR) is the most widespread, extensively known and studied. ASR is the internal chemical reaction between the hydroxyl ions in the pore water within concrete and certain forms of siliceous material in the concrete aggregate. The reaction is essentially expansive, resulting from the formation of a gel, which can imbibe water, swell and exert pressure which can then crack the concrete. In rare circumstances, such expansion can be as much as 2-3%. ASR, like other alkali-aggregate reactivities, is influenced by a large number of factors, and so typically, ASR, like other reactivities again, progresses slowly so that it is usually some years before the effects of expansion and of internal damage become externally visible.

Because ASR is internal, chemical and expansive, the most direct and visible external evidence of ASR is cracking (Fig. 2). The tensile strain capacity of concrete is only of the order of 150 to 200 µs (microstrain), and rarely exceeds 500 µs. In plain concrete elements such as pavements and dams, therefore, cracking will occur as soon as expansive strains exceed this value. In concrete structures, having reinforcing bars and carrying dead and live loads, further complications arise: such structurs are already stressed and cracked under the loads they carry, although such cracks may never be damaging, and often not wide enough to be visible to the naked eye. Expansive strains of even 0.05% to 0.10%, superimposed over load-induced cracks, can then have deleterious implications, and the long term effects of such internal strains, particularly if they are progressive with time, can be serious (Fig. 3).

The forms and reactivity potential of silica in relation to ASR are complex and difficult to identify and determine. The structure and texture of siliceous components are generally described in terms such as amorphous, cryptocrystalline, microcrystalline and crystalline. Silica minerals occur in a wide range of polymorphic forms, many of which are unstable at atmospheric temperatures. The most common forms of silica include quartz, tridymite, cristobalite, opal and the chalcedony group, which includes a number of varieties of silica composed of minute crystals of quartz with submicrostopic pores. These in turn include chalcedony itself, agate, chert, flint and jasper (8).

The reactivity of the different silica minerals depends on the amount of order in the crystal structure. Opal has a very disordered structure and is therefore the most reactive form of silica (Fig. 4). Unstrained quartz, on the other hand, has a well ordered structure and is normally unreactive. Other forms of silica such as glasses, microcrystalline and cryptocrystalline quartz, strained quartz, chalcedony, tridynmite and cristobalite exhibit moderate reactivity.

Petrographic examination and related thin-section studies supplemented where appropriate by X-ray diffraction analysis, infra-red spectroscopy and other instrumental techniques can give useful information on the minerals present in aggregates but it should always be borne in mind that other characteristics of aggregates such as bulk density, porosity, particle size distribution, amount of reactive particles and the ambient environment, all have profound effects on observed expansions (4). It is also important to note that whilst aggregates may be reactive without expansion, expansion does not occur without alkali-induced reaction.

Whilst cracking is the most direct and visible external evidence of ASR (Figs. 1-4), concrete affected by ASR can also show a variety of physical and microstructural characteristics which can be identified through petrographic examinations (9). These include reaction rims round the periphery of the aggregate particles, voids filled with reaction products, cracks in aggregate particles and loss of bond between the aggregate and matrix. Under certain conditions, the gel can also be seen as unsightly stains, popouts and fluid exudations issuing from cracks and seeping on to external concrete surfaces, and in time this gel may react with lime and appear as white and opaque (Figs. 3 and 5). In moist, protected locations, the external surfaces may also show wet reaction spots. On the other hand, several occurrances are known where there is no visible staining or exudation, because of the nature of the gel, so that absence of such exudations cannot always exclude the possibility of ASR being the cause of cracking (10).

Factors Influencing ASR

Because of the chemical nature of ASR, it is necessary that the following should be present for ASR to occur and cause damage

a) sufficient alkali in the concrete
b) a critical amount of reactive silica in the aggregate
c) sufficient moisture.

The importance of the pressure of sufficient moisture can be readily seen because cracking due to ASR arises from the capability of the alkali silica gel formed during the chemical reaction to imbibe water into its structure, which produces expansive forces and hence leads to tensile stresses. Marked deterioration due to ASR will therefore occur under wet storage, and in real practice, under wet environmental conditions. On the other hand, in mass concrete constructions, however, moisture inherent in the concrete may be sufficient to cause deleterious expansive rection (11). Also, in structural members of relatively large dimensions, most concrete mixes may contain adequate amounts of uncombined mix water to permit ASR to initiate and expansion to proceed (12). The gel may thus still form in relatively dry surroundings, but it needs additional water to expand and cause damage. So moist concrete inside a dry building is probably safe from ASR (relative humidity, RH $<$ 75%); but concrete exposed to rain, sea water or ground water is susceptible to ASR. So structures like pavements, retaining walls, foundations and dams are easily affected by ASR. In general, high temperature together with adequate moisture accelerates ASR, whereas low temperatures slow down the reaction. Relative humidity has also an important influence on ASR (13). A combination of changes in relative humidity and temperature can create ASR expansion at various rates (13).

The term alkali used in connection with ASR generally refers to alkalies of sodium and potassium, and both are present in portland cement. Portand cement is, therefore, by far the most important source of alkalies in concrete, but alkalies may also be derived from other constituents such as mixing water, when sea water is used in this capacity, or from aggregates. In fact, alkali-bearing aggregates can also form a major source of alkali for ASR, and alkali levels much greater than those present in high-alkali cements can be leached out from such aggregate materials, irrespective of whether they are reactive or not (14). Experience with concrete pavements in various parts of the world shows that even when a low-alkali cement is used, deleterious expansion due to ASR can develop if the alkali present in the aggregate can be leached out to participate in the reaction (15).

Since alkalies for ASR can be derived from either the portland cement or the aggregate, the use of a low-alkali•cement cannot by itself guarantee prevention of expansive reactions, although in many cases it may possibly do so. An alkali content of 0.6% or less of sodium oxide equivalent is recognized as a low-alkali portland cement; ASR can generally be avoided by using such cements, or others such as sulphate-resisting portland cement. However, the presence of alkalies from external sources, such as deicing salts, may penetrate the hardened concrete, and can aggravate damage caused by ASR (16-18), (Fig. 6). Recent tests show that alkalies from deicing salts can have quick and devastating effects on hardened concrete undergoing or susceptible to ASR. Much of the damage in bridge structures observed in the UK and elsewhere in the world can be attributed to the penetration of salts into concretes containing very slowly reactive aggregates.

It is generally believed that all aggregates are reactive to some degree when used in concrete; however, many concrete structures perform satisfactorily, even when petrographic examination may show some reaction. It is only when the reaction products expand enough to cause cracking that the reaction can be considered to be deleterious. The most expansive aggregates are those containing metastable silica minerals such as opal, tridymite and cristobalite as well as the volcanic glasses. Chelcedony is less reactive and quarz is far less reactive. However, temperature is a major factor influencing ASR, and even aggregates such as flint, generally considered to be unreactive, can generate deleterious expansions under warm (35-40°C) and humid conditions (19).

Since reactive silica, alkalies and water are all necessary for damage due to ASR to occur, these three requirements can be translated in practical terms to the following factors that influence the reaction:

1) the nature, size and amount of reactive siliceous particles;
2) the alkali content of the cement, and the cement content of the concrete;
3) the amount of alkali that could be leached out of aggregates;
4) sources of external alkali, and
5) ambient environmental conditions, i.e. availability of moisture and temperature.

Like all deleterious effects on concrete, concrete quality and particularly its porosity has a considerable influence on both the extent of the ASR and the mode and degree of the resultant damage. Mix proportions and water-cement ratio are thus important factors influencing ASR and, like drying shrinkage, the size of the member may also be influential.

It is clear that the involvement of such a large number of factors in ASR makes the identification and prediction of the possibility of the reaction extremely difficult. Many of the parameters listed above can be controlled or catered for satisfactorily. However, two factors of importance stand out: ambient environment and time. Firstly, some aggregates that are normally relatively unreactive can show deleterious expansion at temperatures of 35-45°C, and there are many ordinary structures which could be exposed to such temperatures at some stage in their life. Secondly, since a combination of favourable factors is required to initiate ASR, expansive reaction can occur at any time in the life of a structure. Often, the effects of ASR do not become evident for several years or even decades, and apparently safe structures begin to show signs of distress with time. Thus, ASR can be serious if and when it occurs, and at the present time there is no test that can guarantee the non-reactivity of a given type of aggregate under all posible constructional conditions.

EXPANSION OF CONCRETE DUE TO ASR

As the expansive strains in concrete due to ASR are the primary cause of structural stress, it is the rate of reaction and the total amount of expansion that are critical to a concrete structure. Both these depend on the particle size, volume and type of reactive aggregate and the various factors listed earlier (Fig. 7). In practical terms, for a given type of rreactive aggregate and cement, both the cement content of the concrete and the ambient environment influence the rate and amount of reaction (Figs. 7 and 8) (19, 20). The data shown in Figs. 7 and 8 have many practical implications to concrete structures. They emphasize the large number of factors involved in ASR.

Flint aggregates are relatively harmles under normal conditions, but can create expansions of 0.10-0.15% under a favourable environment (Fig. 7). On the other hand, with a moderately reactive aggregate, disruptive expansions can occur even with as low an alkali content of concrete as 2.5 kg/m^3 (Fig. 8). These data show the uncertainties inherent in postulating a critical or total alkali content of the concrete as a criterion for potentially deleterious reactivity in real structures. Such limits can obviously provide guidance to designers, but engineers need to bear in mind the sensitivity to deformation of the concrete structure and the actual expansion. Where structural deformations are critical, expansions as little as 0.05% can be excessive, whereas in a well designed and properly reinforced concrete structure with good and correct detailing and anchorage of all steel bars, the detrimental effects of ASR at higher expansions can be readily contained. For concrete constructions which are particularly vulnerable to ASR because of their size, location or exposure, the level of alkali required to minimise the risk of ASR damage may be well below the generally considered 'safe levels'.

EFFECT OF ASR ON CONCRETE PROPERTIES

Apart from the actual magnitude of expansion, the effects of ASR on the engineering properties of the concrete are important to the enginer. Unfortunately, both the rate of expansion and the total expansion depend very much on the reactive aggregate, cement type, cement content and the environment, so the effect on engineering properties cannot be generalized. However, some idea as to how the strength and elastic properties of concrete are affected by ASR is presented in Table 1 which shows the effect of two different types of reactive aggregates - a rapid and highly reactive aggregate (opal) and a slow and moderately reactive aggregate (fused silica) (21).

The results clearly illustrate the significant losses in strength and elastic modulus through ASR. It is important to recognize that these losses do not all occur at the same rate or in proportion to the expansion undergone by the ASR-affected concrete. The data in Table 1 show that, depending on the type of reactive aggregate, loss in compressive strength can vary from 40 to 60% whereas loss in tensile strength can be as high as 65-80% (Fig. 9). The loss in dynamic modulus can also be high, ranging from 60 to 80% (Fig. 10). Thus, the two major properties affected by ASR are flexural strength and elastic modulus, both of which affect the flexural rigidity of a structural member.

The data in Table 1 and Fig. 10 also show that losses in pulse velocity occur with increase in expansion. Indeed, careful analysis of the data shows that both dynamic modulus and pulse velocity, like flexural strength, are highly sensitive to changes occurring in the internal structure of the concrete due to ASR (Fig. 10) (21). All these properties show measurable losses at an early age with low expansions and no visible cracking. These properties can therefore be used to monitor structural deterioration due to ASR.

One important implication of the data in Table 1 is that since losses in engineering properties do not occur at the same rate or in proportion to expansion, it would be dangerous to specify the same limiting values of expansion for all types of structures. Critical limits of deleterious expansion obviously need to be defined according to the type and use of a concrete structure, and limits such as those given in ASTM C227 may need modification if excessive damage in a field structure is to be avoided (22).

ASR IN CONCRETE STRUCTURES

The interrelationship between laboratory studies, conducted whether at microstructural or engineering level, and the field performance of real concrete structures affected by ASR is extremely complex, and influenced by many parameters, the effects of some of which may be contradictory. At a fundamental level, alkali-silica reactions are hydration reactions in which the suspect silica particles, alkali, calcium and hydroxyl ions in solution all participate. The intensity of the reaction, and the severity of its effects would very much depend on the conditions that facilitate the migration of the reactants in the pore liquid in the concrete and water towards the reactive silica in the aggregate particles. In real concrete structures, the construction processes and the surrounding microclimate conditions of heat and moisture may lead to varying and variable local concentrations of reactive silica and concrete alkalinity due to the highly unpredictable in situ material porosity, permeability and diffusivity characteristics. Uniform and homogeneous reactions and structural distress thus occur seldom. Nevertheless, laboratory studies have an invaluable role to play, and are critical to the correct assessment and evaluation of the state of distress in real structures affected by ASR, and in prescribing effective and durable remedies.

When considering ASR in real structures, three significant factors need to be borne in mind. First, ASR is time-dependent, and it occurs invariably when the structure (whether it be a pavement, retaining wall, dam or bridge) is in service, i.e. when the affected structure is already stressed and cracked. Secondly, alkalies from outside sources such as ground water and deicing salts may also penetrate into the concrete, and their effects may be much more damaging than that arising from cement and other alkali sources present at the mixing stage. Thirdly, concrete structures are exposed to a variable environment - an environment that is subjected to seasonal and yearly variations in temperature and humidity.

The net effect of all these three complex and interactive factors is that both moisture gradient and alkali ion transport mechanisms will be non-uniform and non-homogeneous. Thus in real structures there will nearly always be a concentration gradient of alkalies, and alkali salts cannot be assumed to be uniformly distributed throughout the mass of the concrete mix. Further, when alkali salts permeate from an external source or when local drying and evaporation take place at certain parts of the structure, the local concentration of alkali salt may exceed an assumed critical value. This may give rise to alkali silica expansion even though the overall alkali content may be lower than the critical value. Expansive strains are thus superimposed on load-induced stresses and cracks, leading to structural distortion, displacement, and ultimately, unacceptable distress.

Because of the nature of the expansive reaction, and the internal stresses it creates, cracking due to ASR in concrete is random and very irregular; and in plain concrete elements it occurs in all directions in the form of what is known as map cracking (Fig. 2). In reinforced concrete, on the other hand, cracking occurs parallel to the main reinforcing steel - thus, in flexural members ASR cracks appear as horizontal cracks (Fig. 11a) whilst in compression, the cracks occur vertically (Fig. 11b). It is readily seen that cracking due to ASR is very different from cracks due to, for example, drying shrinkage or thermal stresses which occur in the vertical direction in a concrete wall. The expansive reaction also creates large tensile stresses in the reinforcing steel, and the compressive stress developed in the adjoining concrete as a result of the restraint of the bar favours cracking in the axial direction parallel to the main steel.

In the laboratory tests, and in elements where the concrete member is free from external stresses, the cracks are generally a few millimetres wide and several millimetres deep (Figs. 4 and 6). Field examination of many ASR affected structures, on the other hand, shows that crack due to ASR can be many millimetres wide and very deep (Fig. 12) - often deep enough to extend beyond the cover of the reinforcing and prestressing steel. Careful measurements of cracks in a wide range of ASR affected concrete structures including buildings, bridges and walls show that crack widths up to 15mm and crack depths up to 300mm have been observed (7, 23) and in general, wider cracks tend to penetrate deeper.

The degree of cracking and expansion obtained from tests on cores drilled from various concrete structures damaged by ASR and containing various proportions of bronzite andesite, one of the most widely occurring reactive aggregates in Japan, provided further confirmation. Expansive strains of even 0.05 to 0.10% superimposed on stressed and cracked structural elements can thus be deleterious and affect their long term ability to carry loads, particularly if the expansions are progressive with time. Apart from the unsightliness of large cracks, large unexpected and non-uniform structural deformations can result from widespread cracking, leading to distortions, displacements and local failures as have been observed in dams, pavements, parapet walls, retaining walls, bridge elements and similar structures (5-7, 10, 23-26).

Visible cracking due to ASR can be further complicated if the aggregates are slowly reactive, and the reaction needs both moisture and favourable temperature for it to be initiated and to continue. Thus the interior columns of an exposed bridge, sheltered from direct sunshine and rain, may show no cracking whilst the exterior columns may develop extensive cracking. On the other hand, a structural member, partly sheltered and partly exposed by the nature of the structure, may show very different crack patterns, with extensive cracking on the exposed faces and with little or no cracking on the sheltered regions (26).

As with drying shrinkage, the presence of steel reinforcement controls ASR cracking. Detailing, or rather, bad and inadequate detailing of reinforcement can therefore have a predominant influence on ASR cracking in the same way that relatively lightly-reinforced members would be unable to offer adequate restraint and control the expansive strains in concrete. The effects of bad or incorrect detailing in an ASR-affected structure can be readily recognised by irregular cracking, observed for example at the free ends of structural members or at the corners of abutments (26-28). With mildly reactive aggregates, a structure may thus show relatively no cracking except at locations where bad detailing has left the concrete exposed, unable to counteract the expansive strains of ASR. Cracking due to ASR in real structures can thus be highly complex, very variable and unpredictive. It is very much dependent on the amount of expansion, care and efficiency of reinforcement detailing, nature of the structure and the stress distribution in the structure.

EFFECT OF ASR ON STRUCTURAL BEHAVIOUR

The most direct and primary effect of ASR in concrete is the development of expansive strains. It is these strains that subsequently cause cracking and result in losses in strength and elastic modulus. These changes in the integrity and properties of the concrete will affect its flexural stiffness of the concrete member and will subsequently lead to distortions and displacements in the member and structure as a whole. It is important to know how these material and property changes due to ASR influence the structural deformations and performance of reinforced concrete members.

To evaluate the effects on structures, it is important to be able to quantify the maximum expansion that is likely to occur for a given reactive aggregate and concrete mix proportions under the exposure conditions likely to be met in the life of the structure. For a given size of member, mortar bar tests are not the best method for quantitative evaluation of the expansive potential of a given aggregate. Tests show that expansive strains measured in mortar bars instead of on concrete prisms give unrealistically high or unreliable magnitudes and rates of strain development for similar conditions of cement alkalinity, reactive aggregate and environment (20). Concrete prism tests are much more realistic, but even here there may be some size effects and the influence of reinforcement to be considered when assessing expansive strains in structural members from laboratory tests on concrete prisms (4).

Reinforced Concrete Beams

Since cracking, flexural strength and elastic modulus are the properties most affected by ASR, and since these properties influence greatly flexural stiffness, it will be flexural members such as reinforced concrete beams that will show the most distortions when affected by ASR. Experience with ASR in field damaged structures also shows that steel reinforcement offers a significant restraint to expansive strains; thus, it is the compression face remote from the tension steel that will suffer the largest expansive strain and this wil be a maximum when the compression face is unrestrained by the presence of steel (26-28).

To illustrate this, the behaviour of singly reinforced concrete beams with adequate shear reinforcement and containing two different types of reactive aggregates - a highly reactive opal and a slow and moderately reactive fused silica - is considered. Figure 13 shows the concrete strains over the depth of a beam whilst Fig. 14 portrays the resulting strains in the steel reinforcement. Figure 13 also shows the concrete expansive strains in a control unreinforced 75x75x300mm prism for comparison purposes.

These results emphasize several important implications for structural members:

1) There appears to be a size effect on the expansive strains. For the size of beams used in these tests, the maximum measured expansive strain is only about 80% of the expansion shown by a laboratory control prism of similar concrete.

2) The maximum expansive strain occurs at the unrestrained edge: hence, such edges and corners are highly susceptible to cracking as shown in Figs. 3 and 11a.

3) The concrete expansive strains are irreversible.

4) There is a strain gradient over the depth of the member, and the maximum gradient will occur when the compression face is unrestrained. The gradient also varies with the degree of reactivity.

5) The presence of steel reinforcement exercises a controlling effect on the expansive strains.

6) The expansive concrete strains also induce large, irreversible strains in the tension steel (Fig. 14). The steel stress due to such strains may be of the order of 220 MPa for expansive strains of 0.50%, although the steel stresses are not proportional

to the expansive concrete strains.

7) The concrete strain gradients and the high steel strains create a prestressing effect and result in hogging deflections.

The effect of these strains and stresses on structural damage can be monitored by ultrasonic pulse velocity techniques as shown in Fig. 15. In real structures, these strains and stresses will also be superimposed on load-induced strains and cracking. Assuming a tensile stress in steel of about 250 MPa in a concrete structure under service loads, even moderate expansions of 0.1% to 0.2% can substantially increase the steel stress to levels leading to undesirable and irreversible structural deformations.

The large expansive strains and the differential expansive strains create a large number of horizontal cracks, approximately parallel to the tension steel (Fig. 16). The cracks are predominant towards the unrestrained edge, and less in the vicinity of the tension steel, and the anchorage zone. Thus, in a well designed and correctly detailed beam, ASR should not cause any undue distress related to bond or anchorage. Observations of cracking due to ASR in flexural members of real structures confirm this mode of cracking. Further, in reinforced concrete members affected by ASR, the worst expansions and cracking occur at the external exposed surfaces, whilst the effects are progressively reduced towards the core.

Effect on Serviceability and Ultimate Behaviour

There are thus four possible major effects of ASR on reinforced concrete beams, all of which may or may not show at the same time, and which are likely to be enhanced when the beams are non-uniformly reinforced and badly detailed (26-28).

1) excessive concrete tensile strains at unrestrained surfaces, corners and edges
2) high tensile steel strains
3) extensive cracking
4) large deformations/distortions.

All these can result in considerable loss of stiffness and affect adversely the serviceability behaviour of reinforced concrete elements as shown in Table 2. Since the deformations shown in Table 2 are due to ASR alone without the effect of external loads, the potential for structural distress and steel corrosion in real structures can readily be appreciated.

The effect of ASR on ultimate flexural strength is shown in Table 3. With a highly reactive aggregate, the loss in ultimate strength can amount to about 25%. With a moderately reactive aggregate having a total expansion of 0.50-0.65%, the loss in flexural strength is about 15%. The hogging induced by ASR, and the pre-stressing effect arising from the hogging, had obviously beneficial effects and reduced the losses which could have been higher. In a symmetrically doubly reinforced beam, the hogging and pre-stressing effect would not exist, and there would be thus no beneficial effect in terms of strength recovery.

Figures 13-14 and Tables 1 to 3 emphasize the structural distress that could be caused by ASR, and the resulting implications on serviceability behaviour and ultimate strength. Because of the hogging, the ASR-affected beams would show no sagging deflection under external load, and there will be few visible signs of ductility prior to failure. Nor would they show any distress in bond, anchorage or shear either during the progress of ASR or the load test to failure. Thus, whilst ASR could induce an apparent lack of ductility at failure, a correctly detailed reinforced concrete beam should not suffer failure in shear, bond or anchorage.

Reinforced Concrete Columns

The major difference in the visually observed behaviour of ASR-affected reinforced concrete beams and columns is in the nature of their cracking (Fig. 11). In columns, cracks occur in the vertical direction, largely parallel to the reinforcing bars; in the field, these cracks are generally few, and often quite wide (Fig. 11 (7, 23, 26, 29)), whereas if the reaction is allowed to proceed slowly and more uniformly over a period of time, as in laboratory columns, numerous cracks tend to occur as shown in Fig. 17.

The effect of ASR on concrete and steel strains in a full size unloaded column undergoing ASR is shown in Fig. 18; comparable concrete strains in a control prism without reinforcement are also shown in the same figure (28, 29). The results are striking and significant. The concrete expansive strain in a reinforced concrete compression member is very much less than that shown by a control prism of the same concrete; in this case, the expansion of the concrete in the column is only about a third of that of the prism. Far more important is the fact that steel reinforcement exercises a beneficial influence in reducing the concrete strains, much in the same way as it does in reinforced concrete beams (Fig. 13). The role of steel reinforcement near the exterior surfaces in controlling disruptive expansions due to ASR in reinforced concrete structural members is thus of fundamental importance to design engineers. The cracking observed at the free ends of structural members and at the corners of abutments observed in real structures (23, 26) thus bear evidence to the controlling effect of steel reinforcement on the expansive strains due to ASR.

The other important message of Fig. 18 is that, as in reinforced concrete beams (Figs. 13 and 14), the concrete and steel strains arising from ASR are largely irreversible, although some relaxation of these strains due to creep could occur in favourable conditions. However, subsequent wetting and drying can initiate or halt further expansion (Fig. 18). Apart from the fact that moisture is an essential ingredient for the continuation of ASR, this behaviour has three significant implications: firstly, if a structural member contains reactive aggregates, the reaction could be initiated at any time in the life of the structure, under favourable environmental conditions; secondly, continuation and discontinuation of structural distress can proceed progressively until such distress becomes unsafe; and finally, once started, the effects of reactivity cannot be stopped unless specific precautions are taken to contain them.

In the particular case shown in Fig. 18, with moderately reactive aggregates, stresses of the order of 100 MPa can occur in the steel reinforcement due to ASR alone. In reality, the stresses due to external load and the load transfer from concrete to steel in compression members should also be taken into consideration in design.

Figure 18 emphasizes another important phenomenon in ASR affected compression members - the concrete and steel strains are nearly the same. In other words, the bond between concrete and steel remains largely unaffected by ASR, and this is a very promising feature which ensures composite action of the column in real life situations.

In practice, columns are seldom axially loaded, and the effects of eccentric loading occurring together with ASR should be recognised (26, 28, 29). In such members, the steel at the compression and tension faces would show different behaviour because of the structural nature of the columns, the reduction in ultimate strength capacity of an ASR-affected column is only of the order of 10 to 15%.

Figures 13, 14 and 18, and Tables 1-3 illustrate clearly the implications on structural elements of expansive strains occurring in concrete due to ASR. Obviously the effects of these expansive strains would depend on the type of structural member, as well as the degree of chemical reactivity, the location and volume of reinforcement and the environment. The resulting strucutral distress can be superficial, or serious, depending upon the nature of the expansion, but the fact remains that structural engineers need to be aware of the influence of such expansive strains on the behaviour of not only individual members but of the structure as a whole.

REMEDIAL MEASURES

In a short paper it is not possible to deal with remedial measures that could be undertaken to counteract the effects of ASR in all conditions and in all types of concrete and concrete structural members. In particular, the methods of controlling the effects of ASR in an existing structure, and upgrading its serviceability and/or strength are complicated, and are not discussed in this paper. However, the data presented here clearly point to how new construction can be designed to counteract the deleterious strains arising from ASR, and this aspect is briefly discussed.

Control of ASR Through Concrete Mix

Currently there is sufficient evidence to show that there are three possible methods of controlling deleterious expansion caused by ASR in hardened concrete. These are (a) use of non-reactive aggregate, (b) use of low-alkali cement, and (c) incorporation of pozzolanic or mineral admixtures in the concrete. In principle, all these alternatives should offer remedial solutions to a serious practical problem. However, current aggregate reactivity tests cannot exclude potentially reactive aggregates (30), and several examples exist to show that low-alkali are not immune to reaction with alkali-sensitive aggregates. On the other hand, real long-term benefits can be derived through the use of pozzolanic admixtures such as fly ash, slag and silica fume. Their use in concrete provides the most positive method of controlling ASR expansion and the generation of concrete tensile strains (31-36).

However, in spite of considerable research carried out on the role of pozzolans and of siliceous mineral admixtures in controlling ASR expansion, it is fair to say that there are still several aspects of the mechanism of pozzolanic reactions and their control of expansion that are not yet clearly established (37-42). There are also many differences of opinion and conflicting results about the role and effectiveness of mineral admixtures in controlling the effects of ASR. Alkalies in fly ash and slag are considered by some to contribute to the reaction; the effects of silica fume appear to be highly variable, while sometimes fly ash and slag are considered to be not only ineffective but even hazardous! (43-59). All these data clearly show that mineral admixtures do not all control the material damage and structural distortion due to ASR at the same time. What is conclusive, however, is that supplementary cementing materials, when correctly used and at the required amount, have a very positive role in controlling effectively and substantially the expansive strains, cracking, loss of strength, loss of stiffness and structural distortions arising from ASR (60).

Some typical data are presented below to illustrate the role and effectiveness of fly ash, slag and silica fume in controlling the effects of ASR. Table 4 presents data on the effectiveness of one particular fly ash in controlling ASR expansion in unreinforced concrete, whilst Fig. 19 shows the effectiveness of fly ash in controlling reinforcing bar steel strains in beams affected by ASR (60-63). It is seen that whilst fly ash can reduce the concrete and steel strains due to ASR substantially, it cannot completely eliminate the expansive strains. Fly ash is also seen to be able to restore the compressive strength of ASR-affected concrete to that of control contrete; however, the same effect is not reflected in the elastic modulus.

The effectiveness of mineral admixtures in controlling the effect of ASR in reinforced concrete beams, slabs and columns is shown in Tables 5 to 7 respectively. In the beams shown in Table 5, two different types of reactive aggregates were used - a very rapidly as well as highly reactive aggregate, opal, capable of inducing concrete expansive strains of 1.6 to 1.7%, and a slowly but moderately reactive aggregate, a synthetic fused silica, with ability to cause concrete expansive strains of 0.60 to 0.65%. A class F low calcium fly ash was used at two replacement levels of 30% and 50%. The data show that with both types of reactive aggregates, 30% fly ash replacement was inadequate to reduce the ASR expansive strains to acceptable engineering levels. With 50% replacement, the concrete and steel strains due to ASR were reduced substantially to values at which they were unlikely to lead to unacceptable structural deformations. The results of Table 5 in terms of upward deflection, cracking and ultimate strength confirm this.

Table 6 shows that fly ash, slag and silica fume were able to restore the strength of ASR-affected slabs comparable to that of the control slab, even though none of the mineral admixtures were able to eliminate ASR expansive strains completely. What all these data confirm is that mineral admixtures counteract the material and structural effects of ASR at different rates and to varying degrees, and that even though they may not eliminate all the material and structural deterioration arising from ASR completely, they are very effective in restoring material and structural stability.

Control of ASR Through Detailing

The significance of the data presented in Tables 5 and 6 and Fig. 19 is this: in many situations, control of expansive concrete and steel strains and consequent cracking are adequate, and acceptable and satisfactory solutions can be achieved, particularly in unreinforced concrete, through the use of fly ash, slag or silica fume in concrete. However, their advantages and limitations should be recognised: in many sensitive structures, additional precautions may be technically and economically desirable to preserve the stiffness, stability and strength of the structure. What this implies is that damaging tensile strains and structural deformations cannot be completely contained by material changes to concrete alone, although the ultimate strength of the members can be restored to their original value by such changes.

However, data shown in Figs. 13 and 18 emphasize that the deformations arising from expansive strains due to ASR can be controlled by steel reinforcement. The provision of secondary steel in addition to the use of fly ash, slag or microsilica thus provides an excellent, fool-proof and positive method of completely eliminating the structural deformations caused by ASR. The effectiveness of the distribution steel will again depend on their amount and location, but the data presented earlier provide incontroversible evidence of the beneficial role of steel reinforcement in suppressing structural deformations due to ASR (26-29, 53, 54, 60, 61).

Data to confirm this further are presented in Fig. 20 and Table 7 which show the steel strain and ultimate failure loads of a number of reinforced concrete columns (95x95x750mm) designed to counteract the effects of ASR (26, 29). The data in Table 7 show that the loss in strength can be fully controlled, whereas Fig. 20 shows that the disruptive strains due to ASR can be brought down to acceptable engineering levels. Thus the use of mineral admixtures such as fly ash, slag and silica fume in conjunction with additional distribution steel seem to be the route to design to control and contain structural distress arising from ASR.

STRUCTURAL DESIGN FOR ASR

The results presented in this paper thus clearly establish that the way to control structural deformations is to adopt a dual approach, namely, the use of mineral admixtures such as fly ash, slag or silica fume in concrete with or without secondary steel reinforcement.

Figures 21 and 22 present schematic detailing for typical structural members such as beams, columns and pile caps which can effectively and efficiently control the deleterious effects of ASR on structural members. Obviously both approaches are essential only in those members where large expansive strains are expected. In the majority of members, where only moderate structural deformations are expected, the use of mineral admixtures or additional steel should be sufficient to prevent disruptive structural distress. A critical factor in designing for ASR is correct and adequate detailing. A member with well-detailed steel should be able to prevent secondary failures in shear, bond and anchorage. Good detailing with fly ash, slag or silica fume concrete should enable a great majority of structural members to perform satisfactorily at all load levels, even if they are to be affected by ASR at some stage in their service life.

CONCLUSIONS

Alkali-aggregate reactions are recognised to be able to cause cracking and disruptive deformations in plain and reinforced concrete. This paper makes a critical assessment of the material and structural implications of alkali silica reactivity (ASR) with an emphasis on the strength, stiffness and safety of real structures, design and methods of counteracting the deleterious effects of the reactivity. The mechanics of ASR, the factors influencing such reaction, the effect of the engineering properties of concrete exposed to such reaction and the resulting structural deformations and loss in strength are critically discussed. It is also shown that the disruptive effects of ASR in plain and reinforced concrete can be controlled through the use of mineral admixtures such as fly ash, slag and silica fume or/and reinforcement detailing. The role and effectiveness of both the admixtures and secondary steel are critically evaluated. Data are presented to show that the use of supplementary cementing materials, secondary reinforcement and good detailing are the key factors in designing structures able to resist the expansive strains due to ASR, and their effects on concrete.

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Table 1 Effects of ASR expansion on concrete properties.

Property	Mix	Age (days)			
		1	7	28	365
Expansion (%)	a - control	0.0	0.0	0.003	0.021
	b - 4½% opal	0.0	0.071	0.316	1.644
	c - 15% fused silica	0.0	0.0	0.023	0.623
Compressive strength, MPa	a - control	26.7	48.6	60.1	73.5
	b - 4½% opal	31.1	44.4	44.5	27.5
	c - 15% fused silica	-	-	52.5	44.5
Indirect tensile strength, MPa	a - control	2.61	3.58	3.90	4.29
	c - 15% fused silica	-	-	3.29	1.83
Modulus of rupture, MPa	a - control	3.52	4.88	52.25	5.58
	c - 15% fused silica	-	-	4.58	1.30
Dynamic modulus of elasticity, GPa	a - control	35.6	41.0	42.5	45.4
	b - 4½% opal	33.9	32.7	20.8	10.4
	c - 15% fused silica	-	39.5	40.8	18.9
Ultrasonic pulse velocity (km/s)	a - control	4.28	4.60	4.67	4.78
	b - 4½% opal	4.12	4.02	3.48	2.70
	c - 15% fused silica	-	4.57	4.61	3.64

Table 2 Serviceability deformations due to ASR in reinforced concrete beams.

Beam	Concrete expansive strains, μs Compu. face	Concrete expansive strains, μs Tensile face	Steel stress MPa	Max. crack width, mm	Upward deflection, mm	Pulse velocity km/s
Control	50*	190	6	Nil	Nil	4.82
ASR-opal	12,880	1170	450	1.05	6.0	4.52
ASR-fused silica	5,180	850	220	0.40	4.0	4.61

* shrinkage value

Table 3 Ultimate flexural strength of ASR-affected beams.

Beam	Failure load kN	Strength loss %
Control	40,50	-
ASR-opal	30,06	26
ASR-fused silica	34,43	15

Table 4 Effect of fly ash on properties of ASR-affected concrete.

Mix*	Expansion (%)		Compressive strength MPa		Dynamic modulus GPa	
	28 days	1 year	28 days	1 year	28 days	1 year
Control	30	210	60.1	73.5	42.5	45.4
ASR Concrete	230	6230	52.5	44.5	40.8	18.9
30% PFA	100	2480	60.2	66.5	44.4	31.8
40% PFA	140	2380	60.3	-	44.6	36.6
50% PFA	80	1320	60.5	72.9	44.5	39.6

* Reactive aggregate: fused silica. Low-calcium fly ash: Class F.

Table 5 Effectiveness of mineral admixtures in RC beams.

Fly ash replacement %	Opal reactive aggregate					Fused silica reactive aggregate				
	Expansive strains*		Upward defl. mm	Crack width mm	Ult. strength kN	Expansive strains*		Upward defl. mm	Crack width mm	Ult. strength kN
	Concrete	Steel				Concrete	Steel			
Control	50	30	Nil	Nil	40.5	50	30	Nil	Nil	40.5
ASR - 0	12,900	2,340	6	1.05	30.1	5,200	1,100	4	0.40	34.4
30	4,400	840	3	0.15	39.2	2,200	880	2	0.12	38.1
50	230	280	0	Nil	39.9	990	440	1	0.09	38.9

* Microstrains

Table 6 Strength characteristics of ASR-affected slabs with mineral admixtures.

Test series	Expansion %	Concrete strength, MPa		Slab ultimate load kN
		Compression	Flexure	
Control - Series A1	Nil	52.5	4.6	10.9
ASR - Series B1	0.732	41.2	1.7	9.5
ASR + PFA - Series C1	0.136	51.3	4.5	10.4
ASR + slag - Series D1	0.311	53.8	3.0	10.6
ASR + MS - Series E1	0.256	63.9	5.9	11.5

Table 7 Control of loss of strength in RC columns due to ASR.

Column		Ultimate load (kN)	
C1	Control	339	
C2	ASR (FS)*	308	- 9%
C3	50% slag	306	-10%
C5	Additional links	314	- 7%
C4	0.8% S Fibre	330	- 3%
C6	MSW mesh	363	+ 7%
C7	HYW mesh	376	+11%
C9	HYW mesh plus 30% PFA	391	+15%

* Fused silica reactive aggregate. HYW, high yield wire; MSW, mild steel wire; PFA, fly ash; S, steel.

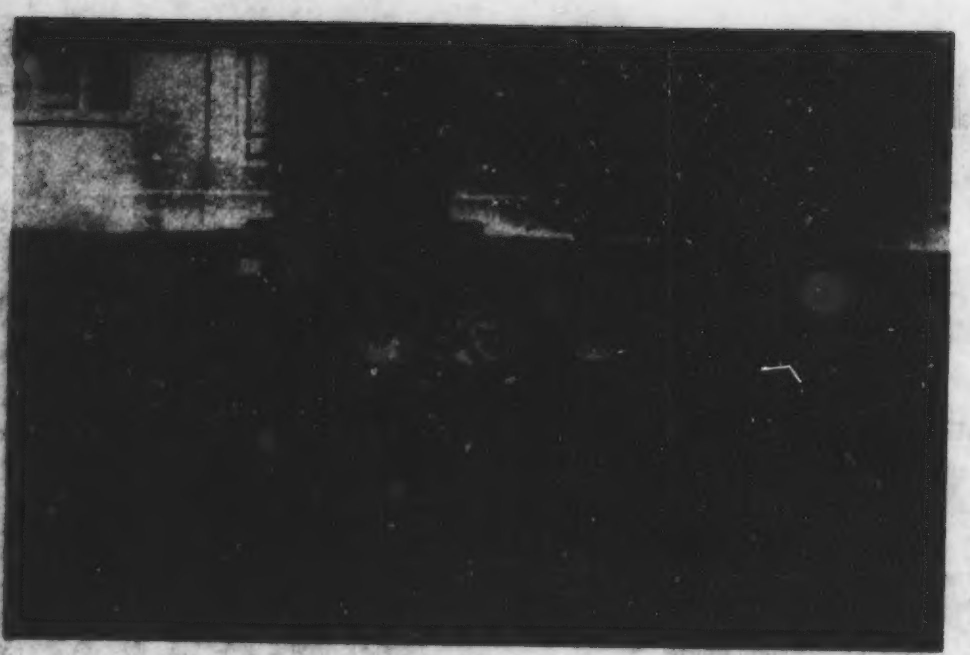


Fig. 1. Airbase pavement failure (New Zealand) twenty years after construction due to restraint of AAR expansion when some new concrete was laid abutting one end.

Fig. 2. Cracking in plain concrete retaining wall (Japan).

Fig. 3. Cracking in a bridge viaduct (Japan).

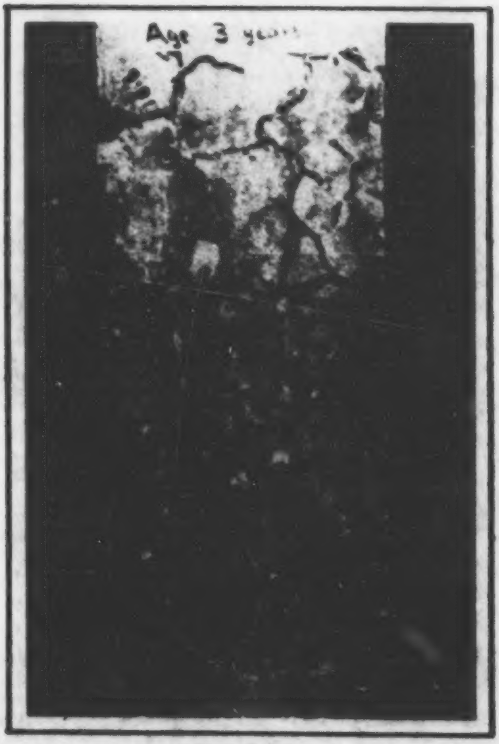


Fig. 4. Cracking and spalling of concrete containing opal reactive aggregate after 3 years.

Fig. 5. Abutment of highway bridge (Japan) showing efflorescence due to solution and precipitation of calcium hydroxide in cracks due to ASR.

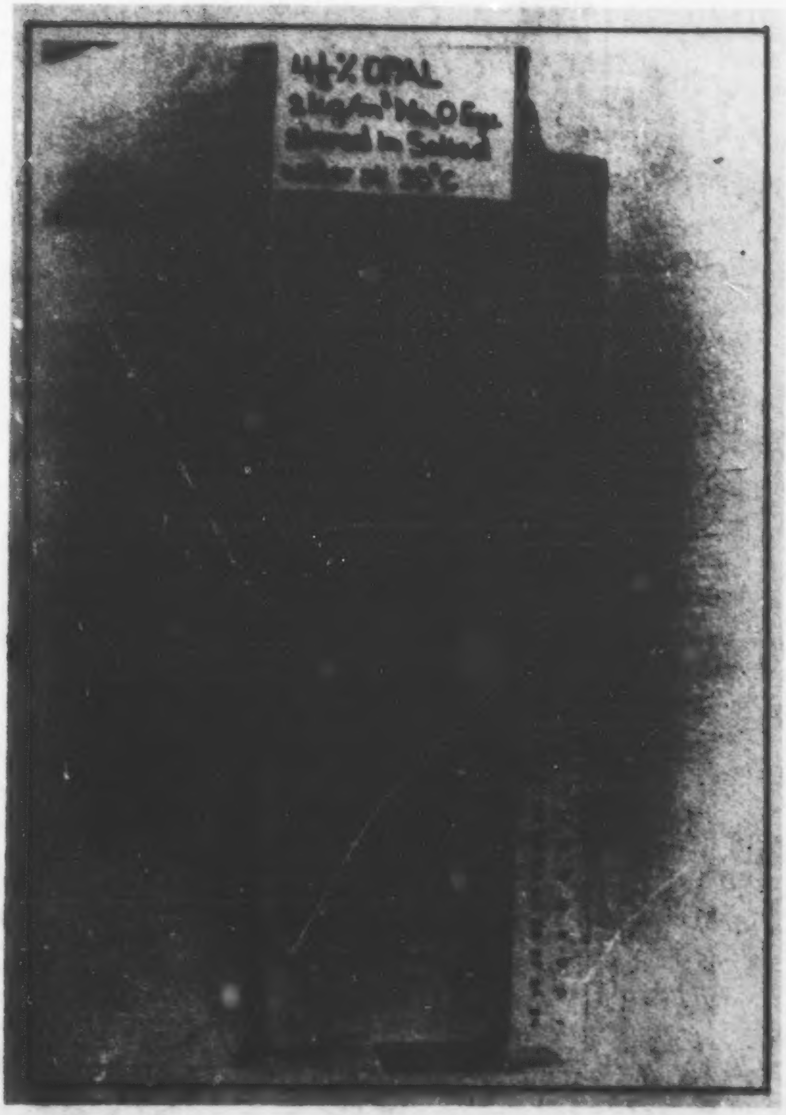

4½% OPAL
2 kg/m³ Na₂O Eq.
Stored in Salted
water at 38°C

Fig. 6. Influence of salt on failure of concrete containing opal reactive aggregate: alkali content 2.0 kg/m^3 sod. ox. equiv., stored in salt solution at 38°C.

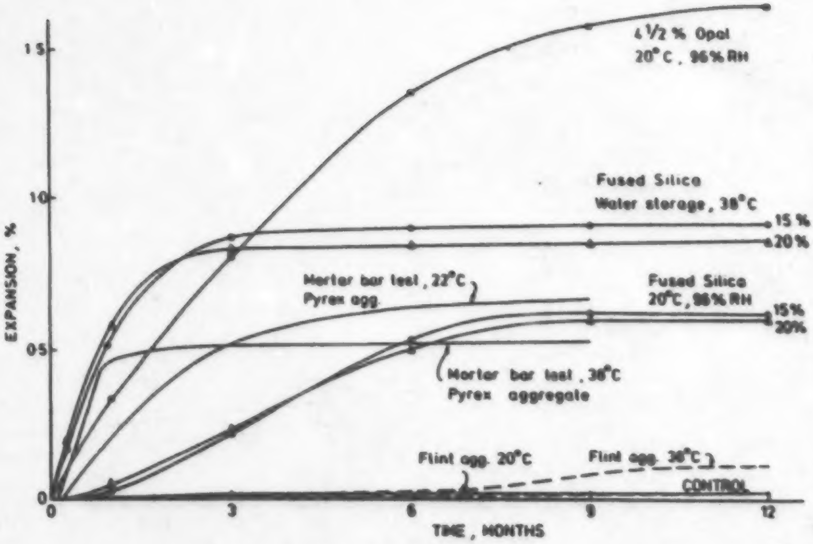


Fig. 7. Influence of type of reactive aggregate and temperature on concrete expansion due to ASR.

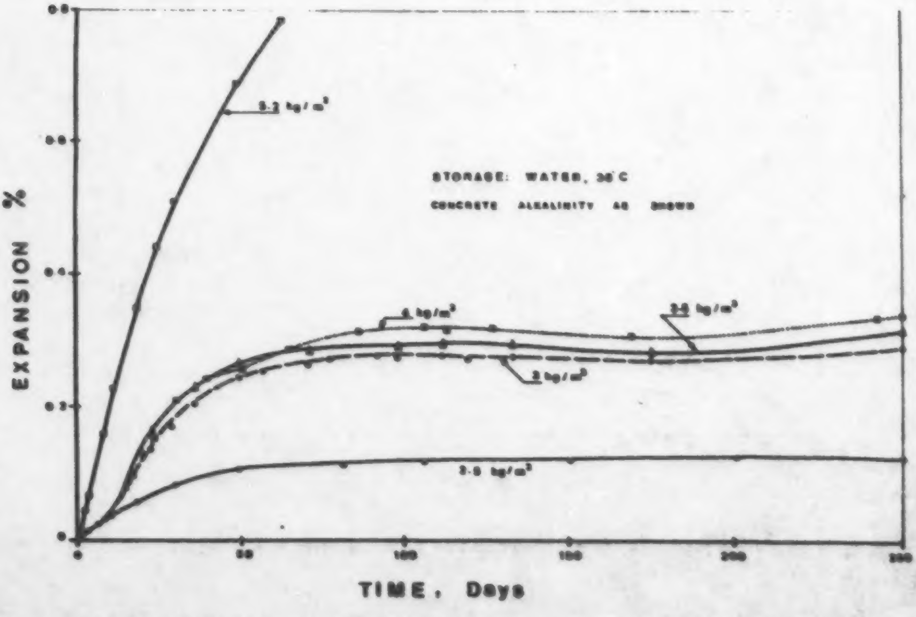


Fig. 8. Effect of concrete alkalinity, i.e. cement content, on ASR expansion.

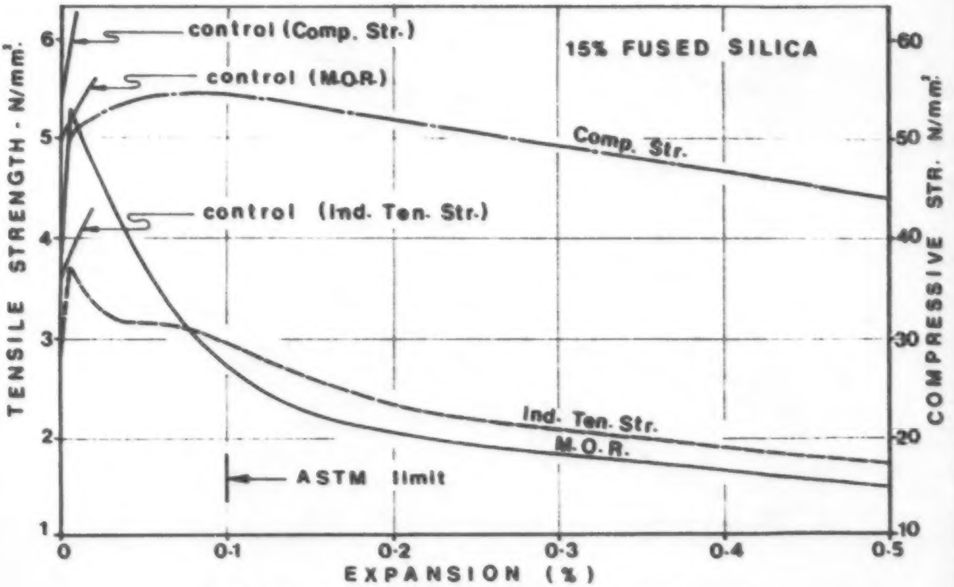


Fig. 9. Loss in compressive and tensile strength due to ASR.

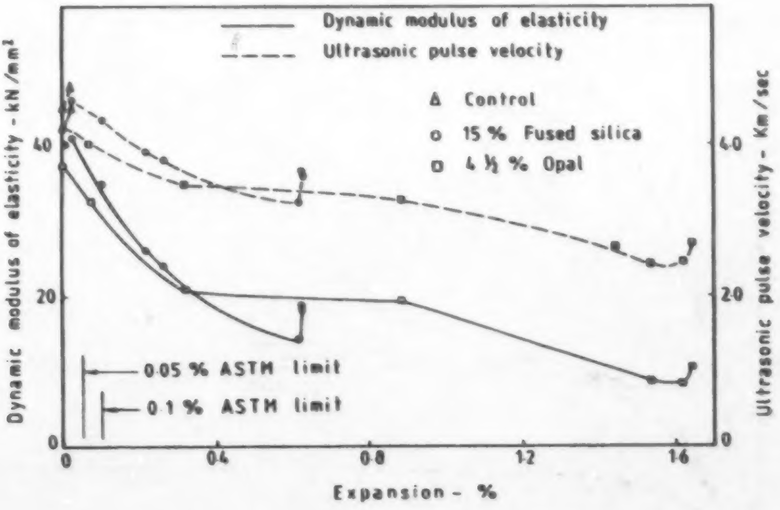
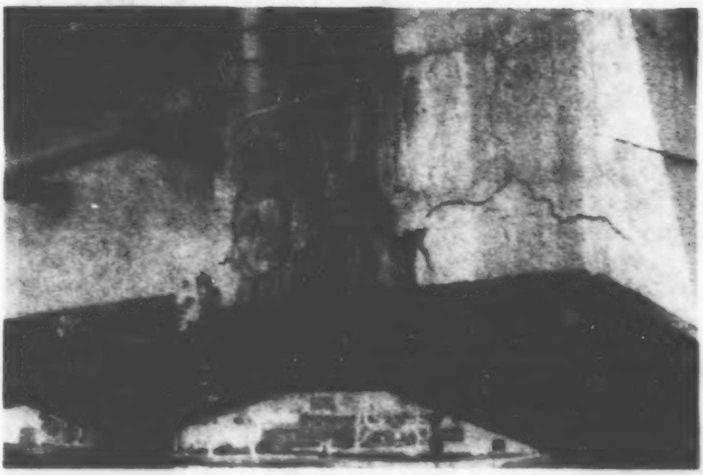


Fig. 10. Loss in elastic modulus and pulse velocity due to ASR.



11(a)

11(b)

Fig. 11. ASR cracking in (a) flexural member (beam of box culvert); (b) compression member (bridge column).

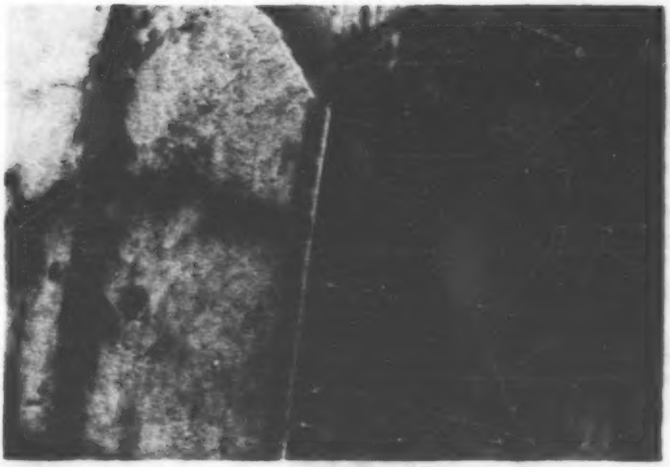


Fig. 12. Wide cracks due to ASR.

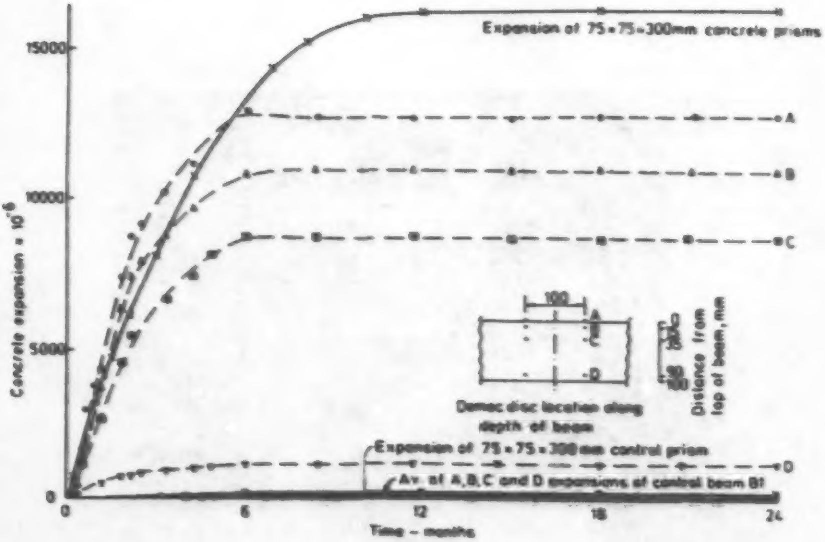


Fig. 13. Concrete strain distribution in an ASR - affected reinforced concrete beam (opal reactive aggregate).

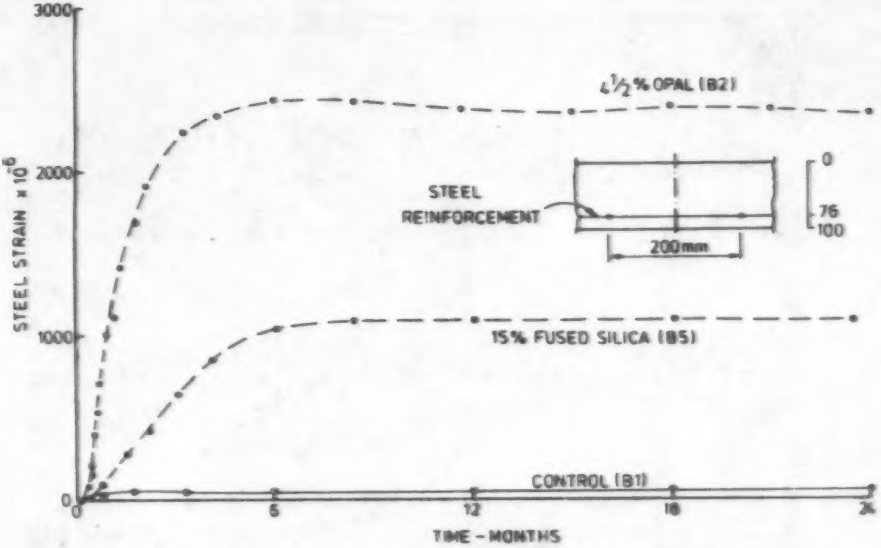


Fig. 14. Development of reinforcement bar strain due to ASR.

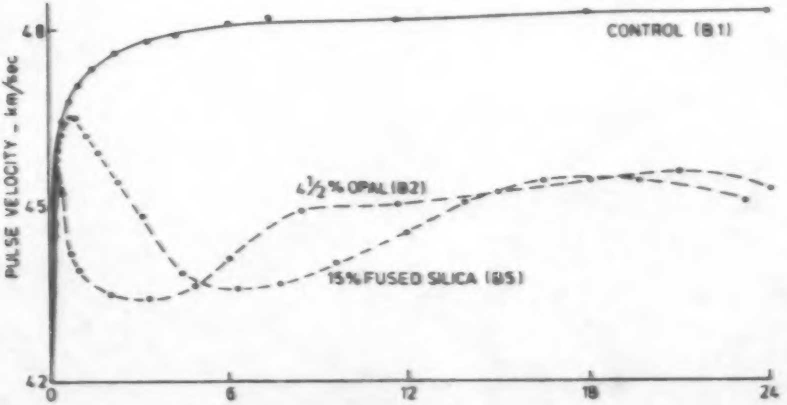


Fig. 15. Longitudinal pulse velocity in ASR-affected beams compared to that in control beams.

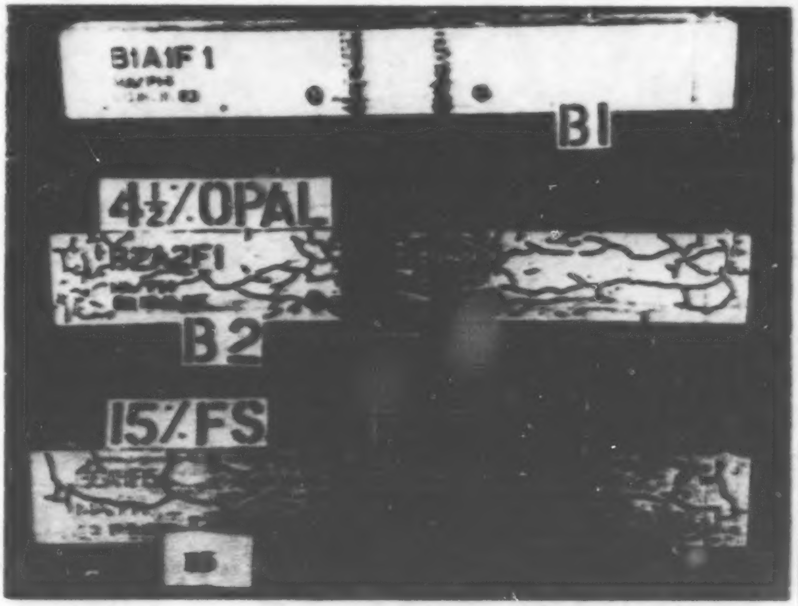

B1A1F 1
B1
4½% OPAL
B2A2F1
B2
15% FS
B5

Fig. 16. Cracking in ASR-affected reinforced concrete beams (a) control beam B1; (b) beam B2 with opal reactive aggregate; (c) beam B5 with fused silica reactive aggregate.

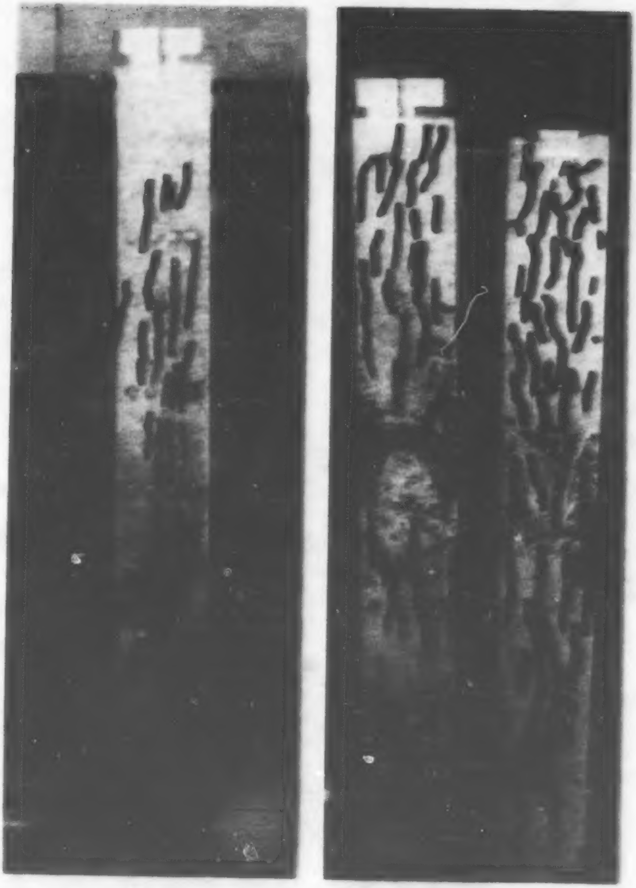


Fig. 17. Cracking in ASR-affected reinforced concrete columns (a) prior to failure; (b) after failure.

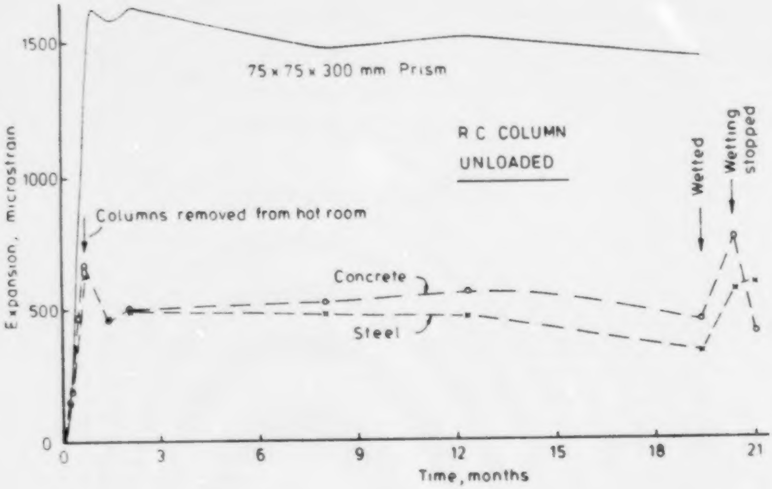


Fig. 18. Concrete and steel strains in an unloaded ASR-affected RC column with comparable concrete strain in a control prism without reinforcement.

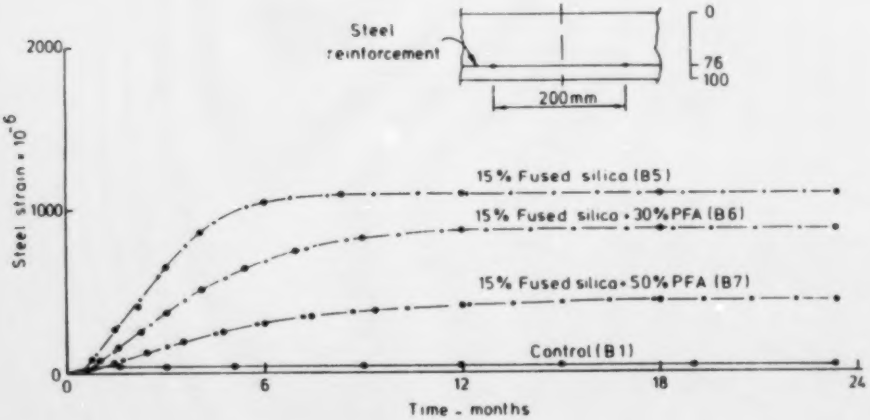


Fig. 19. Steel strains in reinforced concrete beams without and with fly ash and undergoing ASR.

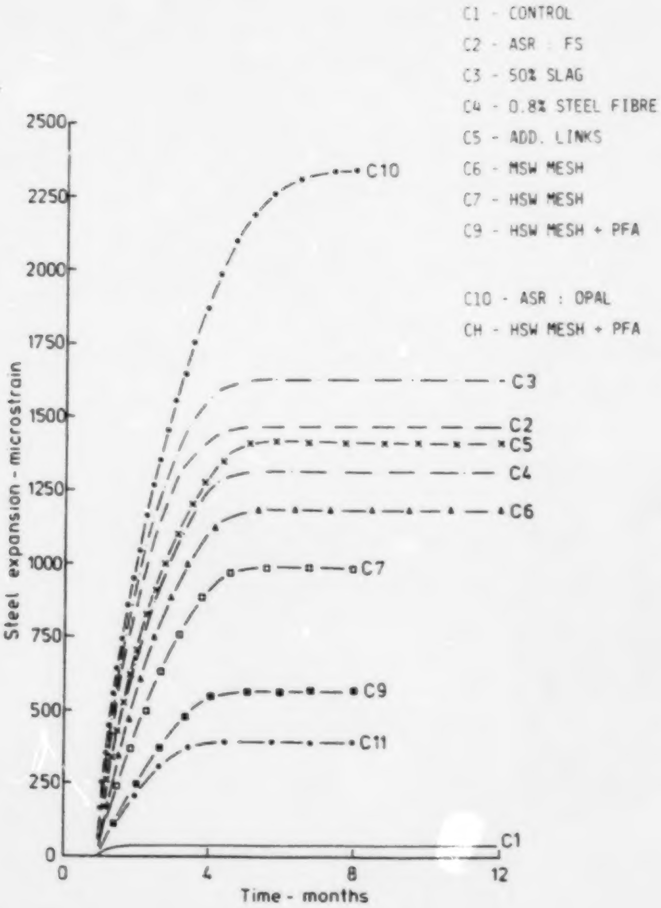


Fig. 20. Control of steel strains in 95 x 95 x 750 mm reinforced concrete columns affected by ASR.

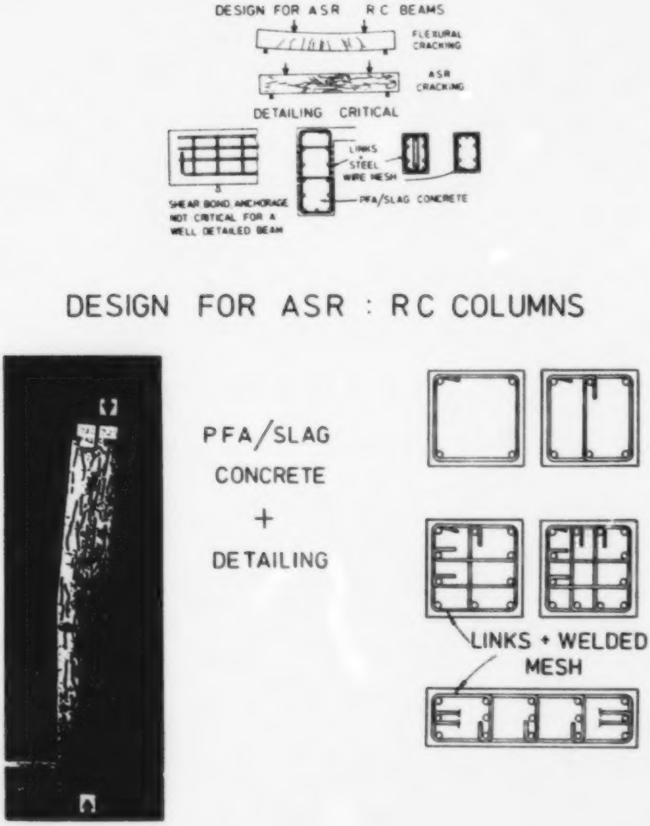

DESIGN FOR ASR RC BEAMS
FLEXURAL CRACKING
ASR CRACKING
DETAILING CRITICAL
LINKS + STEEL WIRE MESH
PFA/SLAG CONCRETE
SHEAR BOND ANCHORAGE NOT CRITICAL FOR A WELL DETAILED BEAM
DESIGN FOR ASR : RC COLUMNS
PFA/SLAG CONCRETE
+
DETAILING
LINKS + WELDED MESH

Fig. 21. Schematic material and structural detailing for control of ASR in (a) RC beam; (b) RC column.

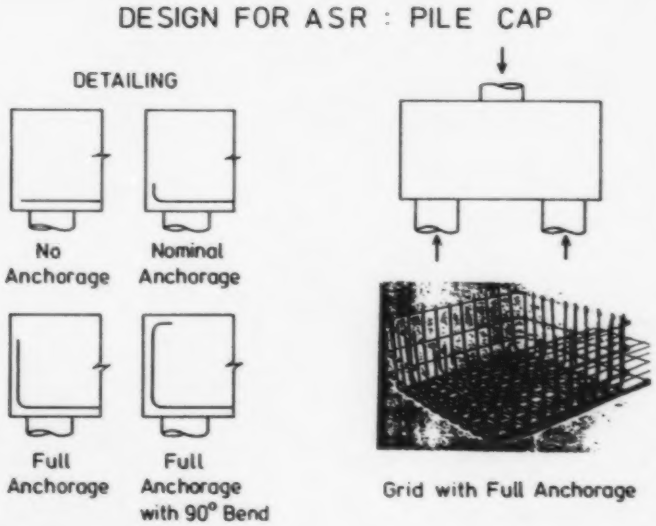

DESIGN FOR ASR : PILE CAP
DETAILING
No Anchorage
Nominal Anchorage
Full Anchorage
Full Anchorage with 90° Bend
Grid with Full Anchorage

Fig. 22. Schematic detailing of control of ASR in pile caps.

Accelerated Test Methods for Alkali-Aggregate Reactivity

by M.A. Bérubé and B. Fournier

Synopsis: The most realistic information on the potential reactivity of concrete aggregates is provided by their field performance in existing structures. However, if such information is not available or judged non reliable for a number of reasons, the aggregates have to be tested in the laboratory. Moreover, on many occasions, they must be evaluated very quickly before construction; this calls for testing methods that are rapid, reliable, simple and reproducible. The paper reviews the principal methods proposed in the literature. In general, they all gave satisfactory results at least with the limited number of aggregates with which they have been developed, but many of them need more investigation before to be adopted as a routine test, while others proved to be not reliable with aggregates from other countries and/or with different composition.

At the present state of knowledge, only a few methods are considered statistically dependable enough to be applicable to most types of concrete aggregates. The Petrographic Examination is always the first step to do. The only other rapid test method that is recommended is the Accelerated Mortar Bar Method (NBRI or ASTM C9-Proposal-P214). This test cannot be used for rejecting materials because it is severe for numerous innocuous aggregates, but remains a powerful screening tool since only a few deleterious aggregates cannot be detected. The most realistic testing method is the Concrete Prism Test performed at 38°C (CAN/CSA A23.2-14A or AFNOR P18-587).

Keywords: accelerated testing methods; alkali-aggregate reactions; alkalies; autoclave; cements; concrete; durability; expansion; mortar (material); testing methods

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INTRODUCTION

Three conditions must prevail to initiate and maintain alkali-aggregate reactions (AAR) in concrete: (1), the aggregates must be reactive; (2), the alkalies must remain abundant in the concrete pore solution; usually, these are mostly supplied by the cement, but some may also be provided by chemical or mineral additives, by some mineral phases present within the aggregate particles such as altered feldspar and zeolites, or by secondary sources such as sea-water and deicing salt; and (3), the concrete must be exposed to high humidity, over 80 to 85% R.H. according to many authors. A critical question thus arises when planning the construction of a concrete structure to be subjected to conditions that might promote the development of AAR: Are the proposed aggregates alkali-reactive in concrete? To answer this question, the most realistic information is provided by the field performance of these aggregates in existing structures. Unfortunately, this information is only available for aggregates frequently used in the past, for many years. Indeed, problems related to alkali-aggregate reactions usually arise years after construction. In the case of hydraulic structures, the above conditions are often not satisfied since there are rarely pretested aggregate sources in the construction area, thus local materials must be used, often the excavated bedrock. The field survey may also be inconclusive for other reasons: (1), limited number of sufficiently aged and/or severely exposed structures built with the aggregates under study or insufficient information about the aggregate sources in the construction files; (2), lack of information concerning the other conditions that affect AAR and the field performance of concrete such as alkali content of the cement, cement content and curing methods; (3), variations in exposure conditions from one structure to another, i.e., humidity, freezing/thawing, wetting/drying, sea-water and deicing salts, and (4), variations in aggregate composition produced by the source between the construction period and the survey of structures such as changes in the exploitation levels or zones, modifications in the methods of exploitation or preparation. For instance, a structure that contains very reactive aggregates might not have suffered from AAR if a low alkali cement or effective mineral additives in sufficient amounts had been used in concrete. Thus, the actual determination of the potential alkali-reactivity of aggregates is often only possible through laboratory tests, which ensure that all aggregates are evaluated under the same conditions.

On many occasions, the aggregates must be evaluated very quickly before construction; this calls for testing methods that are rapid, reliable, simple, and reproducible. In the past few years, intensive work has been undertaken on rapid testing methods for determining the potential alkali-reactivity of concrete aggregates. This paper reviews the test methods, standardized or not, that are presently used or under development. These methods are listed below:

Test Methods for Alkali-Aggregate Reactivity

- **Petrographic Methods** (≥ 1 day)

- **Chemical Methods**
  - ASTM C 289 Chemical Method (2-3 days)
  - Modified Chemical Method (ASTM C 289 on insoluble residues) (2-3 days)
  - German Dissolution Test (1 day)
  - Osmotic Cell Test (< 40 days)
  - Gel Pat Test (≥ 1 week)
  - Chemical Shrinkage Method (1 day)

- **Rock Cylinder Method (ASTM C 586)** (≥ 1 month)

- **Mortar Bar Methods**
  - ASTM C 227 Method (6 months)
  - AFNOR P 18-585 Method (6 months)
  - CCA Method (6 months)
  - Danish Accelerated Method (5 months)
  - NBRI or ASTM C9-Proposal-P214 Accelerated Method (2 weeks)
  - Autoclave Methods (Chinese, Japanese, Canadian) (≥ 3 days)

- **Concrete Prism Methods**
  - CAN/CSA A23.2-14A Method (1 year)
  - AFNOR P 18-587 Method (8 months)
  - South African Method (21-24 days)
  - BSI 812 Method (1 year)
  - CCA Method (6 months)
  - Accelerated Method (used in Quebec) (1 month)
  - Autoclave Methods

First, it must be pointed that all the above methods are accelerated tests, even the CSA Concrete Prism Method, which requires 1 year, and that they cannot exactly reproduce field conditions. Indeed, they all try to simulate in less than one year what may happen in the field after many years. To achieve this goal, it is generally recognized that at least one of the following approaches must be taken: (1), increase the alkali concentration using high-alkali mixtures or immersing the samples in alkaline solutions; (2), store the samples at high temperature, for example 38°C, 80°C, autoclave treatment; (3), subject the samples to high pressure as in the autoclave; (4), subject the samples to high humidity environments such as 100% R.H. or immersion in aqueous solutions; and/or (5), increase the specific area in reducing the aggregates to powder or sand size.

PETROGRAPHIC METHODS

Lists of potentially reactive minerals and corresponding host rock types are available from a number of publications (1-8). The petrographic examination of aggregates in thin sections under the optical microscope usually allows recognition of potentially reactive mineral phases or rocks. Techniques such as X-Ray diffraction (6), measurement of the undulatory extinction angles of quartz grains (3,9), infra-red spectroscopy (10,11), and scanning electron microscopy (7) might be also useful for evaluating the potential reactivity of aggregates. However, Grattan-Bellew (12,13) suggests that quartz with undulatory extinction could not be deleterious, contrary to microcrystalline quartz which is often also present in aggregates containing strained quartz. As pointed by the author (13): "*Petrographic examination alone cannot predict information on the expansiveness of a particular cement-aggregate combination, however, experienced petrographers can predict the likely behavior of aggregates with which they are familiar.*" Indeed, when conducted by a petrographer with experience of AAR, the petrographic examination can sometimes be sufficient to accept or to reject the aggregates under study, in accordance with the past field performance of similar aggregates. If there is doubt, the petrographic examination will help to determine what additional tests to perform, considering the nature of the aggregates under study, thus will prevent poor choices, while reducing the amount of work. Indeed, some testing methods are not capable to detect some deleterious aggregates, while being severe for innocuous ones. In fact, the petrographic examination must always be performed prior to any other test (14,15). This remark holds particularly for rapid testing methods.

CHEMICAL METHODS

Chemical Method ASTM C 289

The ASTM C 289 Chemical Method (16) is certainly one of the most widely used test for evaluating the potential reactivity of silica-bearing aggregates. Its acceptance is mainly because it requires only small quantities of material and results can be obtained within a few days. In this test, 25 g of 150-300 µm particles of crushed aggregate is immersed in 25 ml 1N NaOH solution at 80°C for 24 hours. The solution is filtered and analyzed for dissolved silica (Sc) and reduction in alkalinity (Rc). The results are plotted on a standard graph showing three fields corresponding to innocuous, deleterious and potentially deleterious aggregates (Fig. 1).

This test is essentially applicable to alkali-silica reactivity. Grattan-Bellew (17) showed that the dissolved silica (Sc) values are effective in establishing the relative chemical unstability of the various forms of silica. Sims (18) reported on a linear relationship between the "dissolved silica test" and the proportion of reactive silica in the sample tested. However, a significant number of known alkali-silica reactive aggregates from all over the world pass the test (19,20), while other aggregates with good field performance fail (6,18,20-24). There are several reasons for this and these are as follows:

Selection of aggregates -- The original graph (16) was based on the expansion of the mortar bars, the field performance or the petrographic examination, in some cases, of 71 U.S.A. rocks containing very reactive siliceous mineral phases such as opal, calcedony and volcanic glass, and for which the testing method appears to give apparently clear and unambiguous results. More recently, the test was successfully applied to andesitic rocks from New-Zealand (25) and Japan (26). However, the increased application of the chemical method to aggregates from various countries led to the conclusion that the original chart is not universally applicable (27). Modified decision charts or criteria were often reported based on regional geological considerations, most of them using the values of dissolved silica (Sc) only. Vivian (28) mentioned that, when the amount of dissolved silica exceeds 100 millimoles per litre, the rock or aggregate involved should contain a sufficient amount of reactive material to produce significant expansion in concrete. Thaulow & Olafsson (29) found the 100 millimoles/litre limit useful to establish the potential for reactivity of some Danish and Icelandic sands. Grattan-Bellew (30) reported the following limits for Danish sands containing flint, and Icelandic sands with volcanic glass:

Reactive Aggregate	Non deleterious (mmoles/litre)	Potentially deleterious (mmoles/litre)	Deleterious (mmoles/litre)
Sand with flint	$Sc < 50$	$50 < Sc < 200$	$Sc > 200$
Sand with volc. glass	$Sc < 100$	$100 < Sc < 200$	$Sc > 200$

Brandt & Oberholster (31) suggested to increase the test duration from 3 to 7 days. New criteria were then obtained for potentially reactive aggregates from the Tygerberg Formation: 1% of dissolved silica after 3 days, or more than 2% after 7 days. Sims (32) reported that the chemical method is insensitive to the "pessimum effect". A number of the flint-bearing aggregates used in the UK, such as the Thames Valley flint gravel and sand, were reported to have failed the test while being non deleterious in concrete structures (6,32).

Interference by mineral phases -- Mineral phases such as calcium, magnesium and iron carbonates, hydrated magnesium silicates, gypsum, zeolites, clay minerals, organic matter and iron oxides have proved to create interferences which result in underestimated Sc values due to precipitation of silica or interference during chemical analysis, or in overestimated Rc values due to reactions with Na^+ and OH^- ions (20,33,34). In particular, calcium carbonates lead to precipitation of much of the silica dissolved as CSH, thus causing the acceptance of several reactive aggregates (19,20) (Fig. 1). Also a number of common minerals such as gypsum, limonite, dolomite, ankerite or metabentonite tend to increase Rc (35).

Crushing and sieving effects -- Crushing creates new surfaces through aggregate particles and siliceous mineral phases. These freshly exposed surfaces present more broken siloxane bridges (Si-O-Si) and silanol groups (Si-OH) than particle surfaces that were exposed for a long time in a natural deposit or in a stock pile. Therefore, they are more likely to react and to release silica in the alkaline solution. For instance, many Appalachian lithic gravels, which are crushed for the test, failed the test, while parent sands from the same sources perform well (19). In some other cases, crushing and sieving results in the removal of much of the reactive phases, for instance the reactive quartzitic cement around the non-reactive quartz grains in some Potsdam sandstones (36).

Temperature effect -- The high temperature involved (80°C) may initiate chemical reactions with some mineral phases such as altered feldspars and clay minerals which do not take place at normal temperatures (37).

In many countries, the ASTM C289 Chemical Method proved to be severe for a number of innocuous aggregates due to crushing effect, temperature effect, while being not enough severe for a number of deleterious silicate and carbonate aggregates due to carbonate interference or removal of reactive phases during the preparation of the test specimen. Moreover, modified limit criteria based on regional geological considerations were developped with either limited or good success. Interpreting the results of the Chemical Method necessitates a good knowledge of the petrographical characteristics and the chemical composition of the investigated aggregates, as well as of the limitations of the testing method.

Modified Chemical Method

A modified version of the ASTM C289 Chemical Method has been proposed for evaluating the potential alkali-reactivity of carbonate rocks from the St. Lawrence Lowlands (Quebec) (19, 20,34). It consists in performing the standard test procedure on the insoluble residues of the specimens after complete dissolution in concentrated HCl of the interfering carbonate mineral phases present in the aggregates under study. In Quebec, this method is currently being applied to limestones, dolostones, as well as to carbonate-bearing silicate rocks used in concrete.

Carbonate aggregates -- In the original procedure proposed by Bérard and Roux (34) for carbonate aggregates, a sufficient quantity of crushed aggregate of size 150-300 µm is immersed in concentrated 12N HCl acid until carbonates are completely dissolved. The residue is screened and washed over a 150 µm sieve to retain only the 150-300 µm insoluble fraction. Unfortunately, these particles represent often less than 50% of the overall residue, and a recent study showed a better correlation with the CSA Concrete Prism Method when the modified chemical test is performed on the overall insoluble residue, i.e. 0-300 µm in size obtained by filtration after carbonate acid digestion (20). Three 12.5 g subsamples of dried residue are subjected to the ASTM C 289 test procedure, rather than 25 g, since the insoluble residue of carbonate rocks is in general highly porous. The Sc and Rc values thus obtained are multiplied by 2 to account for the lower mass of material used. However, this correction is not justified and can be omitted. The results are plotted on the ASTM C 289 graph without additional correction and interpreted in accordance with this standard.

The test was applied to 71 limestones and dolostones from the St. Lawrence Lowlands (19,20). When the results obtained are interpreted in accordance with the ASTM C 289 standard, even when the values are not multiplied by 2, 97% of the 35 expansive samples in the CSA Concrete Prism Method, using a limit of 0.04% at 6 months, are classed deleterious or potentially deleterious in the Modified Chemical Method compared to 0% in the Standard Chemical Method, but only 31% of the 36 non-expansive aggregates are found innocuous (19,20). (Fig. 2). The dissolved silica value Sc was found to be the most useful parameter in characterizing the potential reactivity of the aggregates investigated. In addition, the corrected Sc value designated as Sc*[1], was calculated to account for the percentage of insoluble residue in the aggregates under study. Then, all the investigated samples meeting at least one of the following criteria: (1), Sc <100 millimoles/litre; (2), Sc* <10 millimoles/litre; or (3), insoluble residues <6%, were found to correspond to non-expansive aggregates in the CSA Concrete Prism Method (Figs. 3 & 4). All samples exceeding these three criteria at the same time are doubtful. Poor correlations with field performance were obtained for Beekmantown dolostones and Chazy dolomitic limestones, while the application of the above criteria was much successful for Trenton and Black River limestones. In general, aggregates producing Sc values >300 millimoles/litre or Sc* >30 millimoles/litre were found to be positively reactive in the field (19,20).

Silicate aggregates -- Many silicate aggregates contain carbonates in significant amounts. A small amount is enough to produce underestimated Sc values in the standard chemical method. Thus, the modified method is always recommended in these cases, and 25 g sub-samples of insoluble residue can be used. It is recommended to test the overall residue rather than only the 150-300 µm size fraction.

The modified chemical method allows the recognition of well known reactive silicate rocks such as rhyolitic tuffs and chloritic schists from Quebec, which were undetected using the standard test procedure, and which contain small amounts of calcium carbonate (see Fig. 1). However, the modified chemical method is still unable to detect a number of reactive silicate rocks such as some Potsdam sandstones (Fig. 1), while being too severe for a number of innocuous silicate aggregates. For instance, the 39 natural aggregates tested by Mongeau (38), which present large differences in composition and which are well representative of all varieties of sands and gravels used for concrete in Quebec, all meet the requirement of the standard chemical method ASTM C 289, while all fail in the modified method. At the most, only a few of them are deleterious in concrete.

The modified chemical method allows detection of most carbonate alkali-silica reactive aggregates, while the standard ASTM C289 chemical method is not capable. However, it is too severe for a large number of innocuous carbonate aggregates, in particular for the dolomitic rocks belonging to the Beekmantown and Chazy geological groups. Therefore, the method should be used only as a screening tool, and not for aggregate rejection. In particular, the non-reactive aggregates from the Chazy and Beekmantown Groups can be readily recognized by petrographic examination. Moreover, when applied to carbonate aggregates, the modified chemical method becomes more complex and poorly reproducible (20), and should be conducted only by experienced laboratories. This method is not more efficient than the Accelerated Mortar Bar Method, described later.

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[1] Sc* = Sc x % insoluble residue

German Dissolution Test

This test has been developped for evaluating the potential alkali-reactivity of opal and flint-bearing aggregates (6,39). Specified quantities of material with selected grain sizes (1-2 and 2-4 mm) are immersed in a NaOH solution at 90°C for 1 hour. The aggregate fractions are then washed, dried and weighed. The loss in mass, called "soluble to alkalis", is taken as a measure of the potential reactivity of the aggregate.

Similarly to the standard and modified chemical methods described above, the German Dissolution Test is essentially a measurement of the chemical unstability of the investigated aggregate in an alkaline environment, e.g. in a NaOH solution. Consequently, it does not allow the evaluation of particular cements, or reactive aggregate - cementitious material combinations with respect to AAR. It does not indicate how deleterious or expansive an aggregate could be in a concrete structure. In addition, this method does not take into account the contribution of material <1 mm to the potential reactivity of sands (6).

Osmotic Cell Test

The apparatus used in this test (Fig. 5), which partially simulates the interface between aggregate particles and the surrounding cement paste, was developped by Verbeck and Gramlich (40) to study the mechanism of expansion resulting from ASR, and to identify factors that determine whether expansive or non-expansive reaction will occur (41). Since then, it has also been used for identifying the potential for deleterious reactivity of various aggregates (41,42).

The osmotic cell contains two chambers each filled with 1N NaOH solution (42). The chambers are separated by a cement paste membrane (w/c = 0.55). A representative specimen of test aggregate composed of 12.40 g of 150-300 μm particles is placed in the "reaction chamber" (Fig. 5). When AAR reaction occurs, the solution flows from the "reservoir chamber" into the "reaction chamber" (positive flow), through the cement paste membrane. The reactivity of the test aggregate is indicated by the height differencial in the vertical capillary tubes attached to the top of each chamber. Positive flow rates, i.e. the differential as a function of time, of 1.50 to 2 mm per day indicates an aggregate with a potential for deleterious alkali-reactivity in concrete, while reverse or negative flows correspond to non-reactive materials (42). A testing period of 30-40 days is generally sufficient when using 12 g test specimens (42), while 2-3 days are normally sufficient with very reactive opaline material.

Results presented by Stark and co-workers (41,42) indicate that the technique is promising as a rapid method for determining the deleterious reactivity of aggregates. It can also be applied to field concrete investigation in conjunction with other tests.

Gel Pat Test

Stanton et al. (43) and Stanton (44) described a rapid method to assess the potential alkali-silica reactivity of concrete aggregates by gel formation in a strong alkaline solution. In the so-called "gel pat test" (GPT), the selected aggregate particles to be tested are cast in a pat of neat cement. After a convenient curing in a moist room, one surface of the pat is lapped until the particles are exposed. The pat is then immersed, polished face up, in a closed container containing a lime saturated alkali hydroxide solution. The authors reported the method to be highly successful in producing a profuse amount of gel in 5 days at room temperature for the opal-bearing aggregates tested. In addition, for a series of selected reactive aggregates, increasing the conditioning temperature to 54°C produced similar quantities of gel after only 16 to 17 hours, compared to 1 week at 21°C.

The Gel Pat Test has been applied with success in a number of studies. Jones & Tarleton (45) applied a technique somewhat similar to that developped by Stanton et al. (43) to 62 aggregates from various sources in the UK. They found the test useful for reactive materials. However, they emphasized on the fact that if reactivity is shown in the test, this is not necessarily an indication that the aggregate under study will be expansively reactive in concrete. Sims (32) mentioned that: "*Whilst basically a simple qualitative test, the gel pats can furnish much information when employed with repetitive sampling and subjected to regular, detailed and comparative inspections. Furthermore, in polymictic aggregates, the gel pat test enables the reacting particles to be identified*". Poole & Al-Dabbagh (46) and Al-Dabbagh (47) reported the results of the evaluation of potential reactivity of flint-bearing fine aggregates using the GPT at different temperature (20-50°C). They observed that elevated temperature might promote an increased number of particles to react, but deleterious reaction to occur for particles with good field performance. Hobbs (6) described the GPT as a good test for reactive silica that is too finely distributed to be observed under the petrographic microscope. Fournier & Bérubé (48) used a modified version of the GPT (Fig. 6) to evaluate the correlation between the amount of gel formed in this test and expansions measured in the CSA Concrete Prism Method (15) and in the ASTM accelerated mortar bar method (49). They proposed a GPT rating that gives an interesting semi-quantitative evaluation of the potential for deleterious expansion of the investigated aggregate, as well as useful informations on the reaction-expansion mechanisms involved.

The Gel Pat Test was originally designed to evaluate the potential reactivity of opal-bearing aggregates, for which Stanton (44) stated that the test represents a quicker and more reliable way to detect small amount of opal in aggregate samples than the petrographic examination. This test is particularly interesting for aggregate samples containing a variety of rock particles, as it often allows to differentiate between reactive and non-reactive particles (38). However, this method remains qualitative and can be only used as a complementary tool in assessing the potential alkali-reactivity of concrete aggregates. In addition, it does not indicate how potentially deleterious the material tested might be in a concrete structure.

Chemical Shrinkage

Knudsen (50) described a quick chemical test based on the principle that a reactive sand in a concentrated alkaline solution is found to experience a volume contraction called "chemical shrinkage" as a result of dissolution of silica. Since then, this method was accepted as a current testing method for AAR in Denmark, and the test precision has been evaluated through a multilaboratory test program (51).

In this test, 25 g specimens of water-saturated sand of size fractions 0.3-1, 1-2, 2-4 mm are placed in 50 ml flasks filled with a 10N NaOH solution, with precautions taken in eliminating the air contained in the sand. A 0.2 ml pipette is fitted at the top of the flask and the system is then placed in a water bath at 50°C. After ≈20 minutes when thermal equilibrium is reached, the oil miniscus in the pipette starts sinking as a result of chemical reaction or shrinkage. The change in level is recorded for 20 hours. A system called Konometer allows the automatic testing and recording of the results.

The method was applied to 20 Danish sands of various reactivity, and the results were compared with expansions of mortar bars immersed in concentrated NaCl solution at 50°C (52). Using a limit of 0.40 ml shrinkage/kg after 20 hours, which corresponded to one particular sand, the two methods agreed on accepting 5 sands and rejecting 14. Knudsen (50) reported that repeated measurements showed an average relative error of approximately 10% for 20-hour results.

ROCK CYLINDER METHOD (ASTM C 586)

Small rock cylinders, 9 x 35 mm in size, or prisms, 9 x 9 x 35 mm in size, are immersed in a 1N NaOH solution at 23°C, with their length measured periodically (53). The test has been first proposed for detecting alkali-carbonate reactive rocks, according to a limit of expansion of 0.1%, however there is no indication about time in the ASTM standard. Canadian Standards (15) suggest a limit of 0.10% at 4 weeks or 0.20% at 16 weeks to recognize alkali-carbonate reactive rocks, for which the method appeared successful (54). The results are not satisfactory with alkali-silica reactive aggregates. The overall assessment of this test can be summarized as follows:

(1) It is recommended for detecting alkali-carbonate reactive aggregates only.
(2) It is not appropriate for siliceous and silicate aggregates: indeed, the reactive varieties may either: a), expand; b), present no length change; c), show apparent contraction due to the progressive softening of the ends of the specimens, or even d), disintegrate through progressive dissolution of the reactive silica in some rocks, such as the Potsdam sandstones used in the Beauharnois dam (36). The result mainly depends on the presence or not of an available calcium source that allows precipitation of the dissolved silica as

expansive gels, otherwise the silica remains in solution without producing expansion.

(3) For carbonate aggregates, the results are quite inconsistent: the reactive varieties may show highly variable expansion and produce no or abundant siliceous gels.

(4) A large number of specimens is required for the correct evaluation of a given aggregate source or sample.

(5) The results depend on the orientation of the specimen with respect to the structural characteristics of the rocks such as schistosity and bedding (21). The specimens should be always taken perpendicularly to rock bedding (Fig. 7) (55).

(6) Except when testing for alkali-carbonate reactivity, the test has a normal duration of one year; it is thus not attractive in comparison with more realistic expansion tests on mortar or concrete.

(7) The test is not readily acceptable by the Concrete industry because of the small amount of material tested.

MORTAR BAR METHODS

Mortar Bar Method ASTM C 227

Mortar bars are made with the aggregate under study, and a cement containing between 0.70 and 1.10% alkalies, expressed as Na_2O equivalent (56). The bars are stored at 38°C and 100% R.H., in sealed containers, and their length is periodically measured. The expansion limits are 0.10% at 6 months or 0.05% at 3 months, the latter being used only when the 6-month expansion results are not available. According to Grattan-Bellew (13), these limits should be reported at 12 and 6 months, respectively. Such limits are already used by many agencies. Oberholster & Davies (57) even suggested a limit of 0.05% at 1 year.

The test does not apply to alkali-carbonate reactive aggregates (22,58), and also proved to be not capable of detecting many slow-late expanding alkali-silica/silicate reactive aggregates, in particular greywackes and argillites (24,58). According to Hobbs (23), all the reactive aggregates from the UK pass the test. A number of parameters have shown to greatly affect the test results:

Effect of container and wicking -- The test is largely affected by the type of container used, which often differs from one laboratory to another, and particularly by the presence or not of wicks inside the container used (59) (Fig. 8). Indeed, with containers with wicks, for instance the reference ASTM container, a large number of Canadian reactive aggregates satisfy the test requirements (60,61). The presence of wicks promotes leaching of alkalies from the mortar bars, and leads to lower expansions (59). As a result, numerous tests performed in the past and from which it was concluded that the aggregates were innocuous are doubtful if the bars have been stored in containers with wicks inside. Then, when evaluating a given aggregate, it is recommended to test at the same time a reference reactive aggregate which does not release significant alkalies to the pore solution (15,59). Such a procedure has been used by Roy from Agriculture Canada* with the aggregates used in the Gardiner Dam project in Saskatchewan. These aggregates were expanding much less than 0.10% at 1 year but more than a well-known very reactive siliceous limestone tested in parallel.

Effect of alkali content -- Also, the alkali content of the cement used, which can vary from 0.70 to 1.10%, expressed as Na_2O equivalent, may explain experimental variations (Fig. 9). A current practice in many laboratories consists in adding NaOH to the mixture water to increase the total alkali content to 1.25% Na_2O equivalent of cement mass. Such a practice is even recommended by CSA (15) for carbonate aggregates containing chert. The cement/aggregate, in other words the alkali/aggregate, affects the results.

Effect of water/cement -- It has been observed that variations in the w/c** may significantly affect the expansion measured in the mortar bar method. As observed in Fig. 10, with typical reactive limestone aggregates from Quebec, a lower w/c, at least between 0.45 and 0.60, results in higher expansions. A similar trend was also observed by Brotschi & Mehta (62). This behaviour might be attributed to the smaller amount of pore water in mixtures at lower w/c which results in higher concentrations in alkali in the pore solution and consequently increases or accelerates reactions. The lower porosity of low w/c mixtures also offers less room for stocking the deleterious reaction products, while such mixtures are possibly less susceptible to alkali leaching because of a relatively lower permeability compared to high w/c mixtures. The above negative effect of a lower w/c is apparently not counterbalanced by the higher strength normally obtained with corresponding mixtures. On the other hand, with Japanese reactive aggregates, Kishitani et al. (63) obtained less expansion with a water/cement of 0.45 than with 0.50. All these results indicate that the effect of the w/c on the expansive behaviour of mortars made with reactive aggregates is not clear and needs more investigation.

All the above results suggest that: (1), the storage conditions such as the type of container and the wicking; (2), the alkali content; (3), the w/c, and possibly also (4), the cement/aggregate, should be better controlled. In Canada, the CSA concrete method, which does not take much longer, is preferred to the ASTM C 227 Mortar Bar Method since it is considered more reliable. At least, when this mortar test is performed, it is highly recommended: (1), to use a container without absorbent material or wicking; (2), to increase the alkali content to 1.25% Na_2O equivalent of cement mass, by adding NaOH to the mix water, and (3), to control the water/cement to 0.50 but 0.44 for uncrushed natural sands, as recommanded in the accelerated mortar method discussed later.

Mortar Bar Method AFNOR P18-585

Recently, a mortar bar method has been adopted by AFNOR (Association Française de Normalisation (64). The principal features of this test are:

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* Personal communication

** No water/cement is specified in the test method as water is added to reach a specified flow.

- The alkali content of the mortar is increased to 1.25%, expressed as Na_2O equivalent.
- The bars are sligthly smaller than ASTM bars.
- They are stored over water in a sealed metal container without wicking. The container is stored over water inside an isolated "reactor" in which the temperature is maintained at 38°C by heating the bottom water. Up to nine containers can be stored inside one reactor. Under such conditions, the temperature variations would be very low in the containers, thus minimizing condensation and alkali leaching from the bars. As a result, higher expansion could be obtained (65).

Such modifications in mixture proportioning and experimental conditions were suggested to overcome some of the problems encountered in ASTM C 227. Another step should involve controlling the aggregate/cement and the w/c.

CCA Mortar Bar Method

In the method proposed by the Cement and Concrete Association, three bars, 25 x 25 x 250 mm in size, are cast using an aggregate/cement of 3, a 0.40 w/c, and a cement with an alkali content of about 1.0%, expressed as Na_2O equivalent (6). The bars are individually wrapped with a damp tissue and placed in plastic sleeves. The tissue is moistened periodically. The expansion limit is 0.05% after 6 months. However, an aggregate expanding more is not classed as deleteriously reactive before a petrographic examination in thin section established that expansion is probably due to AAR (6). This method could overcome many of the problems relating to the influence of the w/c, aggregate/cement, alkali content, and the storage conditions in ASTM C 227.

Danish Accelerated Mortar Bar Method

Mortar bars, 40 x 40 x 160 mm in size, are prepared with a cement:aggregate:water ratio of 1:2:0.50 (52). The bars are immersed in water for 28 days, then transferred in a saturated NaCl solution maintained at 50°C, and measured once a week. The proposed limit between innocuous and deleterious aggregates is 0.10% after 20 weeks (29). The Danish method has been applied to a number of Danish, Icelandic and Swedish sands for which a good correlation was obtained with both tests ASTM C 227, using a limit of 0.10% at 1 year, and ASTM C 289, using a limit criterion of Sc >100 millimoles/litre (27). Hooton & Rogers (66) used it with twelve Canadian aggregates with known petrographic characteristics and field performance, however using mortar bars made in accordance with ASTM C 227. These tests failed in recognizing the reactive aggregates tested.

Chatterji et al. (67) showed that in the long term, the mortar bars tested in the Danish Accelerated Mortar Bar Method may expand more than parent bars immersed in NaOH 1N at 50°C, even if much lower expansions are observed in the short term in the former case. The same trend is obtained when testing CSA concrete prisms in NaCl 1M (6%) and NaOH 1N solutions at 38°C (19,68). The total expansion is attributed not only to AAR, but also to the formation of calcium chloro-aluminates hydrates. This has been demonstrated to take place in mortar or concrete samples immersed in NaCl (14,69). Moreover, the Danish method takes 5 months, thus is not significantly faster than more realistic expansion tests on concrete specimens.

Accelerated Mortar Bar Method (NBRI, ASTM C9-Proposal-P214)

Mortar bars for this test, which is known as the NBRI test, are made in accordance with ASTM C 227 and immersed after demoulding in water at 23°C, with the containers immediately placed in an oven stove at 80°C. The next day, the zero reading is taken, then the bars are immersed in a 1N NaOH solution at 80°C for two weeks (12 days in the original NBRI proposal), and measured hot each working day (57). The original NBRI proposal required 12 days of immersion in the NaOH solution.

Davies & Oberholster (70) observed that expansion of mortar bars generally increased when increasing the w/c from 0.47 to 0.55. A similar trend was reported by Fournier & Bérubé (71) for w/c from 0.45 to 0.60, and by Blanchette (72) for mixtures made at w/c of 0.45 and 0.50. The ASTM C9-Proposal-P214 (49) recommends to use a fixed w/c of 0.50 for coarse aggregates and manufactured sands, and 0.44 for natural sands. On the other hand, the type and the alkali content of the cement used have no significant effect on the expansion values, provided that the cement did not contain mineral admixtures (13,70,71,73). Indeed, the use of various cements in a multilaboratory test program on the method did not increase significantly the coefficients of variation obtained with a very reactive limestone (73). In the ASTM C9-Proposal-P214 (49) and CSA (74) proposals, the volumetric ratio between the NaOH solution and the test bars is also controlled between 3.5 and 4.5.

There is a general agreement that aggregates which expand less than 0.10% after 14 (or 12) days be considered innocuous. Indeed, up to now, only a few deleteriously reactive aggregates were found to satisfy this limit: the alkali-carbonate reactive rocks from the Kingston area, Ontario (75), some Potsdam sandstones from the Montreal area (Fig. 11), a particular phyllite from Australia (76) and some granite/gneiss from the USA*. Grattan-Bellew (75) mentioned that it may be necessary to consider different acceptance criteria according to the type of aggregate tested, suggesting 0.10% for reactive limestones, 0.20% for greywackes and argillites, and 0.15% for the other aggregate types. However, the author finally concluded that a 0.15% limit at 14 days could satisfy most of the investigated cases, while stressing the importance of petrographic examination prior to testing. Furthermore, Davies and Oberholster (70) observed slowly expansive siliceous rocks to expand between 0.10% and 0.25% after 12 days, while proposing a limit of 0.25% for rapidly expansive siliceous rocks. The same limits at 14 days are proposed in the CSA proposal (74), while the higher limit is a little lower (0.20%) in the ASTM C9-Proposal-P214 (49).

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* Personal communication by R.D. Hooton, University of Toronto.

Hooton & Rogers (66) reported the results of tests conducted on various Canadian rocks, mostly from Ontario. The authors observed that some aggregates with good field performance, but petrographically marginal, as well as others which appeared slowly reactive in concrete structures, produced expansion between 0.10% and 0.25% in the ASTM C9-Proposal-P214, therefore suggesting further testing for such aggregates. Fournier & Bérubé (77,78) tested a number of limestone and dolostone aggregates from the St. Lawrence Lowlands in Quebec (Fig. 12). They proposed a 14-day expansion limit of 0.10% for non-reactive aggregates, and >0.25% for reactive aggregates. Again, materials producing expansion between 0.10% and 0.25% should require further testing. In this particular study, one marginally reactive aggregate was even exceeding 0.25%, and 19% of the 36 non-expansive samples, in the CSA concrete test, were expanding more than 0.10% at 14 days in the accelerated mortar bar method. Similar results were also reported by Blanchette (72) for a number of carbonate and silicate rocks from Quebec. In the study by Ouellet (36), one reactive aggregate, the Potsdam sandstone, expanded less than 0.10% at 14-day, while 44% of the innocuous or presumably innocuous silicate rocks exceeded this limit (Fig. 11). A high proportion (74%) of the 39 natural sands and gravels from Quebec tested by Mongeau (38) expanded more than 0.10% at 14 days in the ASTM C9-Proposal-P214 test, with 49% of them expanding more than 0.25%, while many performed well in the ASTM C 227 mortar bar method using containers without wicks (Fig. 13). At the most, only a few of these natural aggregates are deleterious in concrete structures. However, one can note on Fig. 13 that all expansive samples in ASTM C 227 were expanding more than 0.25% in the accelerated mortar bar method.

Proposed modifications by other investigators -- Hudec (79) in his modification, uses samples consisting of small cores, 19 to 25 mm in diameter x 70 mm long, taken from mortar or concrete blocks. The cores are put in 1N NaOH at 80°C for 2 days, and then cooled to room temperature to facilitate handling and measurement which are made using LVDT gauges. This cycle is repeated for a minimum of 14, and a maximum of 21 days. According to the author, the expansions obtained are not sufficiently reproducible, but they could be normalized in accordance with the expansions obtained for two reference aggregates tested in parallel, one reactive and one non-reactive.

Yoshioka et al. (69) have suggested the following modification to the test procedure. After curing for 24 hours in their moulds at 23°C and 100% R.H., ASTM C 227 mortar bars are: (1), demoulded and cured for another 24 hours at the same conditions; (2), immersed for one day in NaOH 1N at 80°C; (3), returned at 23°C and 100% R.H. for another day and (4), then re-immersed in NaOH at 80°C for three more days. In addition to the zero reading, the bars are measured at the end of the treatment, after thermic equilibrium is reached at room temperature (23°C). Results can be obtained in 7 days. The authors do not propose limits for expansion but observed good correlation with results from ASTM C 227 mortar bar tests.

The Accelerated Mortar Bar Method should be used with care for rejecting aggregates. Indeed, many innocuous aggregates that performed well in the field and/or in other laboratory tests on mortar or concrete specimens, have been reported to have failed the test when using the proposed limit of 0.10% expansion at 14 days, and even sometimes exceeded 0.25%. Then further testing is required for aggregates expanding more than 0.10% at 14 days, at least for those between

0.10% to 0.25%. As noted by Grattan-Bellew (13), "*the larger expansions obtained with the NBRI method creates a danger that aggregates with a satisfactory field performance might be classified as deleteriously expansive*". Nevertheless, this test remains a useful screening tool as it is capable of recognizing within two weeks most deleterious aggregates (except the alkali-carbonate reactive aggregates and a limited number of other aggregates, such as some Potsdam sandstones), while allowing the acceptance of a high proportion of innocuous aggregates. This test method has been applied to a large number of aggregates in several countries. In addition, the precision of the test method has also been assessed by several workers (13, 70,71,73), with very satisfactory results. The test was also used with success for determining the effectiveness of mineral additives such as silica fume, fly ash and blast furnace slag in the presence of alkali-silica reactive aggregates (80-82). Moreover, Davies & Oberholster (83), Shayan & Quick (84) and Fournier & Bérubé (78) observed that the ASR reaction products formed during this test were similar to those found in ASR affected field concrete.

Autoclave Mortar Bar Methods

Some researchers proposed testing methods for potential AAR which involve steam curing and/or boiling in water or alkaline solutions of mortar or concrete samples under relatively high temperatures and pressures. The dimensions of the samples tested, as well as the mixture proportions used and the procedure adopted, vary from one author to another, but they are quite similar or consist of modified versions of original methods proposed by Tang et al. (85), Tamura et al. (86) and Nishibayashi et al. (87). The potential alkali-reactivity of the investigated aggregates is determined by measurement of length change, or by visual inspection of surface cracking and measurements of internal degradation. These methods generally give reliable results, at least for the limited number of aggregates investigated. They were also used to evaluate the effect of mineral admixtures on alkali-silica reactivity (88), and the future susceptibility to AAR of fresh concrete (89). Most of these methods were recently reviewed by Hobbs (6), Grattan-Bellew (13) and Fournier et al. (90).

The test proposed by Tang and co-authors (85,88) requires two days. Small mortar bars, 10 x 10 x 40 mm in size, are made with a cement : aggregate : water ratio of 10 : 1 : 3 and with an alkali content raised to 1.5% Na_2O equivalent of cement mass, by adding NaOH to the mixture water. The bars are cured for 24 hours at 20°C and 100% R.H., steam-cured for 4 hours, then immersed in a 10% KOH solution for 6 hours at 150°C, in autoclave. Expansions in excess of 0.10 - 0.15% correspond to aggregates that are potentially deleterious in concrete. Unfortunately, the test failed in detecting a number of reactive aggregates, such as an andesite, a rhyolite and a tuff (85). In addition, the authors indicated that an aggregate which is identified as reactive in the autoclave test might not necessarily react deleteriously in concrete. Moreover, the test has only been applied to a limited number of aggregates and further testing should be necessary to confirm the limit criteria to be used with other types of aggregates (13). The principal advantage of this test is that it only requires a small amount of material, so it can be used, for instance, to evaluate the reactivity of individual gravel particles.

Criaud et al. (91) tested a number of reactive and sound aggregates of various types and origin in a slightly modified version of the above test, called the Microbar Method. In this method, cement : aggregate ratios of 2, 5 and 10 are used instead of only one in the original chinese method. The following expansion limits are proposed: (1), < 0.08% correspond to sound aggregates; (2), 0.08% to 0.10% correspond to potentially reactive aggregates, and (3), > 0.10% indicate reactive aggregates. The authors reported that the method provides a reliable evaluation of the potential alkali-silica and silicate reactivity of aggregates, but cannot detect alkali-carbonate reactive rocks. The reliability of the method also seems to be improved by performing the test at 3 cement/aggregate, slowly expanding materials being more effectively detected at lower cement/aggregate.

In a method proposed by Nishibayashi et al. (87), mortar bars, 40 x 40 x 160 mm in size, are made according to a cement : aggregate : water ratio of 1 : 2.25 : 0.45 and with an alkali content raised to 2% Na_2O equivalent of cement mass, by adding NaOH to the mixture water. The bars are cured in their moulds for 24 hours at 20°C and 100% R.H., then placed in autoclave under 0.15 MPa (≈120°C) for 4-5 hours. The authors do not propose acceptance limits but the expansion obtained after five hours in autoclave were almost similar to those obtained after 20 weeks in ASTM C 227 mortar bar tests. So far, the test has been applied only to a limited number of Japanese aggregates and further testing is thus necessary to confirm the limit criteria to be used with other types of aggregates.

In a method proposed by Tamura and co-workers (86,92), mortar bars, 40 x 40 x 160 mm in size, are made according to a cement : aggregate : water ratio of 4 : 2 : 1 and with an alkali content raised to 2.5% Na_2O equivalent of cement mass, by adding NaOH to the mixture water. The bars are cured for 24 hours in their moulds at 20°C and 100% R.H., demoulded and immersed in water at 20°C for another 24 hours, then stored for 2 hours in boiling water in an autoclave maintained under 0.15 N/mm^2 (≈111°C). After cooling at 20°C, the bars are examined for the presence of cracks. The propagation of ultrasonic waves through the bars and the dynamic elasticity modulus (Young) are also measured. The samples are considered reactive when: (1), cracks are observed; (2), the Young modulus decreased by 15% or more; or (3), the acoustic waves are slowed down by 5% or more. This test has been applied to a limited number of Japanese aggregates and further testing should thus be necessary to confirm the limit criteria to be used with other types of aggregates.

In a test proposed by Fournier et al. (90), mortar bars are made in accordance with ASTM C 227, but with a constant w/c of 0.50 and 0.44 for uncrushed natural sands. The alkali content of the mixture is raised to 3.5% Na_2O equivalent of cement mass, by adding NaOH to the mixture water. The bars are placed in an autoclave commonly used for testing hydraulic cement, under 0.17 MPa (≈130°C) for 5 hours. The test was applied to 17 silicate rocks (Fig. 14), 40 carbonate rocks (Fig. 15), and 39 natural sands and gravels (Fig. 16) from Quebec. An autoclave expansion limit of 0.15% was then chosen to differentiate deleterious or potentially deleterious aggregates from non-reactive ones. The efficiency in recognizing the potential alkali-reactivity of the reactive aggregates or the innocuous character of the non-reactive rocks investigated was generally better with this proposed autoclave test than with the accelerated mortar bar method, particularly for the quarried and natural silicate aggregates tested.

METHODS USING CONCRETE SAMPLES

Concrete Prism Method CAN CSA-A23.2-14A

Concrete prisms not less than 75 x 75 x 300 mm and not more than 120 x 120 x 450 mm in size are made with the coarse aggregate under study, a non-reactive sand, and a normal cement containing between 0.8% and 1.2% alkalies, expressed as Na_2O equivalent (15). Since 1986, the alkalies must be increased to 1.25% Na_2O equivalent of cement mass (310 kg/m^3), by adding NaOH to the mixture water. Two storage conditions are proposed: 1), in a humid room at 23°C; and 2), over water in sealed containers maintained at 38°C. The former condition is used for assessing alkali-carbonate reactivity, while the latter can be used, and in fact is highly recommanded, for any rock types. Length measurements are made periodically up to 1 year. The expansion at 1 year should not exceed 0.04% at 38°C (or 0.025% at 23°C). The test is recommended for all types of aggregates (22,30).

Effect of temperature and alkali content -- A temperature of 23°C is sufficient for recognizing alkali-carbonate reactive aggregates. However, 38°C is recommended for the other types of aggregates (15), in particular for the slow-late expanding silica/silicate reactive aggregates, for which the limit of 0.04% seems still reasonable (93). As a result of the modifications proposed in 1986, i.e. higher temperature and higher alkali content, many deleterious aggregates which were not detected in the past, can now be recognized (Fig. 17).

Effect of water/cement and coarse aggregate/sand -- Variations in the w/c, which is not presently controlled in the current standard test procedure, water being adjusted to obtain a slump between 70 and 90 mm, may significantly affect the results. As observed in Fig. 18, with reactive limestone aggregates, a lower ratio, at least down to 0.45, contrary to other findings with opaline materials (94), could lead to higher expansions, as observed previously for the ASTM C 227 Mortar Bar Method (Fig. 10), and for the same reasons. Sprung & Adabian (95) also observed the same trend with greywackes and argillites. Unfortunately, the w/c may vary with some characteristics of the coarse and fine aggregates, such as the particle shape and the absorption. For instance, jaw crushing tends to produce flat particles with some types of aggregates, which in turn leads to higher w/c to reach the specified slump, and likely to lower expansions. Then a marginally deleterious aggregate has more chance to satisfy the test requirements when the test sample contains a large amount of flat and elongated particles. Therefore, the w/c should be controlled when running a concrete test. In practice, it is in most laboratories which currently perform CSA concrete prism tests. In a similar way, the ratio between coarse and fine aggregates, which can vary between 50/50 and 65/35 in the current standard, might affect the results since influencing the ratio between the amount of available alkalies and the number or the surface area of the reactive aggregate particles.

Effect of cement fineness -- The amount of expansion in the concrete prism test could be also a function of the cement fineness. In this respect, Krell (94) obtained increasing expansion with increasing cement fineness. This could be explained by the fact that the alkalies originally present in the cement are released more rapidly in the pore solution, then being more rapidly available for the reactions (13).

Influence of storage conditions -- The results obtained in the CSA Concrete Prism Method are also affected by the storage conditions, i.e. temperature and humidity (59,96,97). In order to counteract this problem, some workers recommend to test in parallel a reference reactive aggregate to control laboratory variations. Indeed, with an alkali-carbonate reactive limestone, Rogers & Hooton (59,96,97) tested four sets of conditions: (1), normal humid room at 23°C; (2), sealed plastic bags containing 100 mL of water, and stored in the same room at 23°C; (3), immersion in a 5% NaCl solution at 23°C, and (4), over water at 38°C in a sealed box with filter paper wicks. Except for the above condition (3), significant amounts of alkalies were leached from the prisms after 2.5 years, e.g. 63%, 22%, and 42%, for conditions (1), (2) and (4), respectively, and the amount of expansion at 23°C was related to the amount of alkalies remaining in the prisms.

So, the storage conditions used in most laboratories, e.g., prisms stored over water in sealed plastic containers with wicks, allow leaching of a large proportion of the alkali initially present inside the specimens; for example 42% after 2.5 years for prisms made with alkali-carbonate reactive agregates. In the long term, the expansion curves tend to flatten out as a result of alkali depletion inside the specimens. On the other hand, a number of experiments suggest that concrete samples containing reactive aggregates and mineral admixtures, in particular condensed silica fumes, may start to expand in the long term (81,82); therefore, it is particularly critical to continue the test until 2 years or more when testing mineral admixtures, while minimizing alkali leaching; otherwise the actual long-term behavior could be misleading.

Experiments at NRC (13) and at Laval University suggest that expansions for prisms stored over water at 38°C, in sealed containers, are similar or even lower when the wicks are removed from the containers (7). So, it seems that high humidity is more critical for concrete prisms than alkali leaching, contrary to mortar bars. Ranc et al. (65) observed sligthly lower expansions with very alkali-silica reactive limestones using wicks but in a different storage system: the specimens are stored over water in a sealed metal container which is itself stored over water inside an isolated "reactor" in which the temperature is maintained at 38°C by heating the bottom water. When used, the wicking material was covering the inner walls of the reactor. The authors also obtained significant lower expansions when the concrete specimens were stored in plastic bags at 38°C, as was also observed at the Ministry of Transportation of Quebec*. A few experiments conducted at Laval University also suggest that the expansions are lower by about 15 to 20% when the concrete specimens are immersed in water 30 minutes before each measurement, as specified in the CSA standard, compared with no immersion (7). This is attributed to some alkali leaching by water (7,19,68).

Effect of the cement content -- The CSA concrete prism method can detect most reactive aggregates except several ones from New-Brunswick (61), including argillites and natural gravels, and also a number of lithic gravel aggregates from Ontario and Quebec (38). According to Rogers (96), in order to recognize the slow-late expanding alkali-silica/silicate reactive aggregates, such as greywackes, argillites, quartzwackes, quartzites, phyllites, arkoses, sandstones, granites with highly strained quartz, the test should be modified to prevent excessive alkali-leaching, for instance by storing the prisms in sealed plastic bags, or to compensate for this phenomena by adding more alkalies or using more cement in the concrete mixture. Based on laboratory test results and known field performance of various aggregates, Grattan-Bellew (75) suggests to use a cement content of 410 kg/m^3 rather than 310 kg/m^3. Mongeau (38) drew the same conclusion for natural aggregates from Quebec.

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* Personal communication by D. Vézina, Transports Québec.

Experimental variations -- The results of a recent multilaboratory study involving twenty different laboratories and alkali-carbonate reactive aggregates from Ontario, suggest that the coefficient of variation for the CSA Concrete Prism Method is about 23% for the four sets of storage conditions described above (96). The lowest average 1-year expansion values were obtained for prisms cured in humid room at 23°C, where alkali leaching was the most effective, while the highest expansion values were obtained for prisms in sealed containers at 38°C. However, the comparison of the results obtained by various workers on the same coarse aggregates suggest that the coefficient of variation could be worse with alkali-silica reactive aggregates (18,19), this calling for another multilaboratory study on such aggregates. This study was initiated by CANMET in 1991.

The CSA concrete prism method currently performed in Canada for evaluating the potential alkali-reactivity of concrete aggregates (15) needs additional adjustments or better controls in order to be more reproducible and to better simulate field performance. It could be necessary to exercise more control on: (1), the w/c rather than the slump; (2), the storage conditions in order to minimize alkali leaching from the prisms, and possibly also (3), the proportion between coarse and fine aggregates. Moreover, the test is not capable of detecting a number of slow-late expanding reactive aggregates for which a higher cement content seems to be required.

Recently, a CSA Task Group worked on the revision of the CAN/CSA A23.2-14A Concrete Prism Method and proposed a number of modifications in order: (1), to overcome the difficulty in detecting the potential reactivity of slow-late expanding reactive aggregates; (2), to minimize alkali leaching from the concrete prisms; and (3), to reduce experimental variations from one laboratory to another. In particular, (1), the cement content should be increased to about 420 kg/m^3 while still raising the alkali content of the mixture to 1.25% Na_2O equivalent of cement mass, by adding NaOH to the mixture water; this corresponds to a total alkali supply from the cement of 5.25 kg/m^3 rather than 3.90 kg/m^3 in the current standard; (2), the water/cement should be controlled between 0.42 to 0.45; (3), the prisms should be individually wrapped in damp tissue and inserted in sealed plastic bags with a little water, then stored over water in sealed plastic buckets with wicks; (4), the proportions between coarse and fine aggregates should be fixed to 60/40, and (5), the cross section of the concrete prisms should be fixed at 75 x 75 mm with the possibility for their length to vary between 285 and 325 mm. Moreover, the test should be always performed at 38°C, even when testing for alkali-carbonate reactivity, and it will be recommended to use the method also for testing sands in presence of a non-reactive coarse aggregate, or coarse-fine aggregate combinations.

Nevertheless, even with the current version, the CSA concrete prism test still remains the most realistic method for evaluating the alkali-reactivity potential of concrete aggregates in the laboratory. The test is capable of recognizing most reactive aggregates. However, it requires one year to conclude, and is therefore not satisfactory in many situations.

Concrete Prism Method AFNOR P 18-587

Recently, a concrete prism method has been adopted by AFNOR (Association Française de Normalisation) (98) which is somewhat similar to the CSA Concrete Prism Method with the following exceptions:

- The cement content of the concrete is increased from 310 to 410 kg/m^3, with the alkali content of the mixture raised to 1.25% Na_2O equivalent of cement mass, thus obtaining a total alkali content of 5.13 kg/m^3.
- The coarse aggregate/sand is fixed to 58/42.
- The prisms are sligthly smaller than CSA prisms.
- They are stored over water in a sealed metal container without wicking which is itself stored over water inside an isolated "reactor" in which the temperature is maintained at 38°C by heating the bottom water. Up to nine containers can be stored inside one reactor. Under such conditions, the temperature variations would be very low inside the containers, thus minimizing condensation and alkali leaching from the prisms. As a result, higher and more progressive expansions are apparently obtained (65).

The modifications concerning the mixture proportioning and the storage conditions were mainly based on the Canadian experience and should improve the reliability of the test. Another step should involve the w/c. Additional data are needed before concluding further on test performance, but this French method is probably the most realistic concrete prism method presently standardized in the world, until the CSA standard is modified.

South African Concrete Prism Method

In this method, concrete prisms, 75 x 75 x 300 mm in size, are made with a coarse aggregate : sand : cement : water ratio of 3.38 : 1.87 : 1 : 0.5 and a cement with a very high alkali content, approximately 1.30% Na_2O equivalent. The specimens are stored over water in sealed containers at 38°C (6). Again, additional data are needed before concluding further on test performance. However, with aggregates from the Tygerberg formation, the results obtained correlated well with those from ASTM C 227 mortar bar tests (31).

Concrete Prism Method BSI-812

In the Draft BS 812 (99), four concrete prisms of 75 x 75 x 200 mm are cast using the coarse aggregate : sand : cement : water volume ratio of 1.73 : 0.74

: 1 : 1.02, and a cement content of 700 kg/m^3. The alkali content is raised to 1.0% Na_2O equivalent of cement mass, by adding sodium sulphate to the mixture water, thus giving a total alkali content of 7 kg/m^3. The test is also recommended for testing sands in presence of a non-reactive coarse aggregate, or coarse-fine aggregate combinations. The specimens are wrapped in damp paper towels, placed in plastic sleeves with excess water and stored above water in sealed containers at 38°C.

Compared to the CSA Concrete Prism Method, the specimen dimensions, the mixture proportions, and the storage conditions are better controlled in this test. However, referring to the very high alkali content (7 kg/m^3) and to particular test results, Grattan-Bellew (13) pointed out that "*excessive increases in the cement content of concrete prisms can however lead to satisfactory aggregates being classed as deleteriously expansive*".

CCA Concrete Prism Method

In this method proposed by the Cement and Concrete Association, in the UK, three concrete prisms, 50 x 50 x 250 mm in size, are made with an aggregate : cement : water ratio of 3 : 1 : 0.40, and with a cement containing ≈1.0% alkali, expressed as Na_2O equivalent (6). The prisms are wrapped in moist tissue and stored in plastic sleeves at 20°C. The expansion limit is 0.05% after 6 months. However, an aggregate expanding more than the above limit is not classed as deleteriously reactive until a petrographic examination in thin section establishes that expansion is probably due to AAR (6).

Again, the specimen dimensions, the mixture proportions, and the storage conditions are better controlled in this test than in the current CSA standard. Moreover, the mixture proportions used correspond approximately to a total alkali content of 5.5 kg/m^3, which is close to the value suggested in the AFNOR method and to the one proposed for the next revision of the CSA standard.

Accelerated Concrete Prism Method

In Quebec, concrete prisms are made in accordance with the CSA Concrete Prism Method, however usually with a constant w/c of 0.50 or 0.55, and cured in a 1N NaOH solution at 80°C. Blanchette (72) suggests a limit of 0.04% expansion after 24 days of immersion in the NaOH solution, which is in accordance with the results obtained at Laval University for silicate aggregates (36) (Fig. 19), as well as for carbonate aggregates (77) (Fig. 20). Such a limit ensures that all the deleterious aggregates tested (8 silicates and 19 carbonates) are classed expansive. However, numerous innocuous aggregates, for instance dolostones and dolomitic limestones from Beekmantown and Chazy Groups from the St.Lawrence Lowlands (72,77), volcanic rocks from the Eastern Townships (36), and natural gravels from the Quebec Appalachians (38), which appear innocuous in field concretes as well as in the normal CSA Concrete Prism Method, are expansive in concrete prisms immersed in NaOH 1N at 80°C. In this respect, the method is inferior to the accelerated mortar bar method ASTM C9-

Proposal-P214. Therefore this accelerated concrete test, as well as its equivalent on mortar, may be an useful screening tool, but it cannot be used for rejecting aggregates; significant expansions only mean that further testing is required. In Quebec (Canada), the accelerated concrete method is commonly applied to core samples for assessing the actual reactivity of aggregates found in existing structures (68).

Autoclave Concrete Prism Methods

Kanazu et al. (100) applied the test procedure proposed by Tang et al. (85) to concrete prisms, 100 x 100 x 360 mm in size. The tests were made at three levels of alkali content, that is 0.67%, 1.10% and 1.5% Na_2O equivalent of cement mass, with the total alkali level being adjusted (from the starting 0.67% level) by adding NaOH to the mixture water. The authors obtained only limited expansion after a preliminary steam curing period of 10 hours at 90°C, while significantly higher expansion values were obtained after 6 hours of autoclave curing at 150°C. They observed that increasing the alkali content over 1.10% permitted to judge the reactivity of the aggregates investigated.

DECISION CHART

In Canada, a number of testing methods such as the Chemical Method ASTM C 289 and the Mortar Bar Method ASTM C 227 proved to be not reliable enough to be adopted as routine tests , while others are very promising, for instance the autoclave mortar bar method developed at Laval University (90). At the present state of knowledge, only three test methods are considered applicable to most Canadian concrete aggregates: (1), the petrographic examination; (2), the ASTM C9-Proposal-P214 Accelerated Mortar Bar Method, and (3), the CSA A23.2-14A Concrete Prism Method. Accordingly, a decision chart, which is, however, mainly based on Quebec experience of AAR and could not necessarily apply to all aggregates found in other Canadian Provinces or in other countries, is proposed on Fig. 21.

CONCLUSION

In general, the most realistic information on the potential reactivity of concrete aggregates is provided by their field performance in existing structures. However, if such information is not available or judged insufficient for a number of reasons, the aggregates have to be tested in the laboratory. In many occasions, they must be evaluated very quickly before construction, which calls for testing methods that are rapid, reliable, simple and reproducible. According to Grattan-Bellew (13), "*all the rapid test methods appear to give satisfactory results, at least with the limited range of aggregates which have been tested*". Among the large number of test methods that have been proposed in the past forty years, many need more investigation. Indeed, before being used as a routine test, any testing method must always be checked by many workers, and with different types of aggregates. It is also clear that none of the proposed methods can be successfully applied to every type of aggregate. Therefore, it is always preferable to perform many different tests.

It must be highly emphasized that the Petrographic Examination (101) is always the first step to do. It can be used to accept or even to reject the aggregate under study, or at least to select the more appropriate test methods to run, in order to prevent poor choices and reduce the amount of work. Indeed, some testing methods are not capable of detecting some deleterious aggregates, while being too severe for innocuous ones. The only other rapid testing method that is statistically dependable and more highly recommended as a routine test is the Accelerated Mortar Bar Method (NBRI or ASTM C9-Proposal-P214). This method cannot be used for rejecting materials because it has been found too severe for a number of innocuous aggregates. However, it remains a powerful screening tool since only a few deleterious aggregates cannot be detected. The Concrete Prism Method performed at 38°C (AFNOR P 8-587, CAN/CSA A23.2-14A, etc.) is considered as the most realistic testing method for determining the potential reactivity of concrete aggregates.

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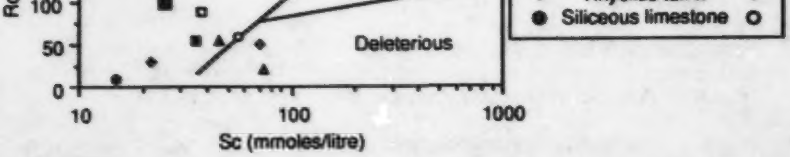


Fig. 1 - Examples of alkali-reactive aggregates from Quebec that are not detected with the standard Chemical Method ASTM C289. However, most of them appear deleterious or potentially deleterious when the test is performed on the insoluble residues (modified method).

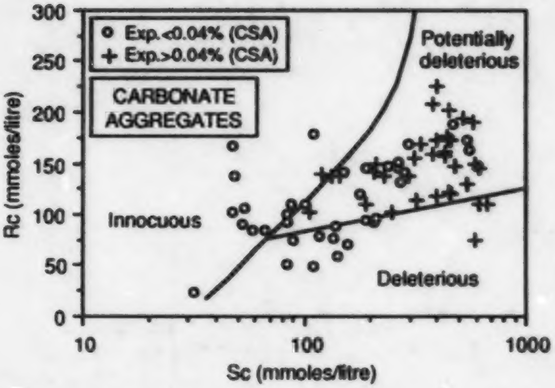


Fig. 2 - Results obtained at Laval University using the Modified Chemical Method ASTM C289 on insoluble residues for a series of carbonate aggregates from Quebec. When the data are plotted on the standard ASTM graph, 97% of the 35 aggregates that induced concrete prism expansion >0.04% are classed deleterious or potentially deleterious, while only 31% of the 36 non-expansive aggregates are evaluated correctly.

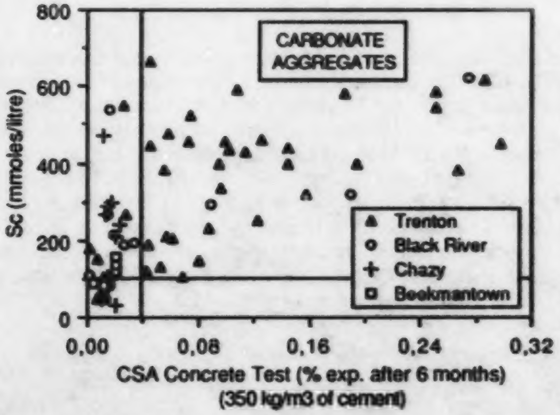


Fig. 3 - Results obtained at Laval University using the Modified Chemical Method ASTM C289 on insoluble residues for a series of carbonate aggregates from Quebec. The amounts of dissolved silica (Sc) are compared with expansions obtained in the CSA Concrete Prism Method. When using a Sc limit of 100 millimoles/litre, all the 35 aggregates that induced 6-month concrete prism expansion >0.04% are classed potentially reactive, while only 36% of the 36 non-expansive carbonate aggregates are classed innocuous (55% for the Trenton Group, 25% for Black River, 33% for Chazy, and 25% for Beekmantown).

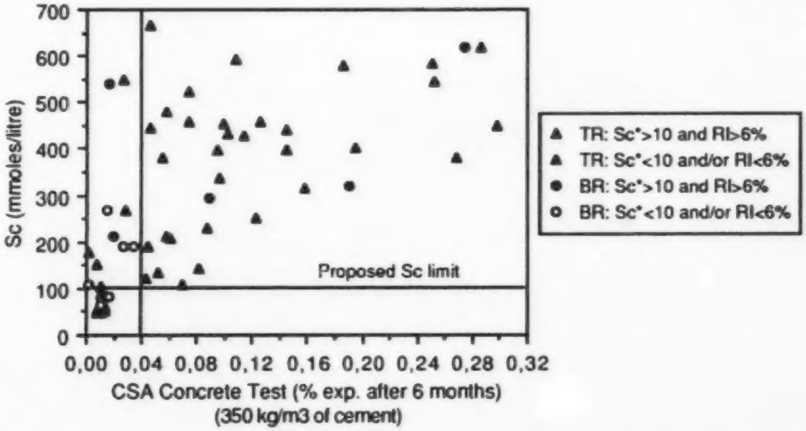
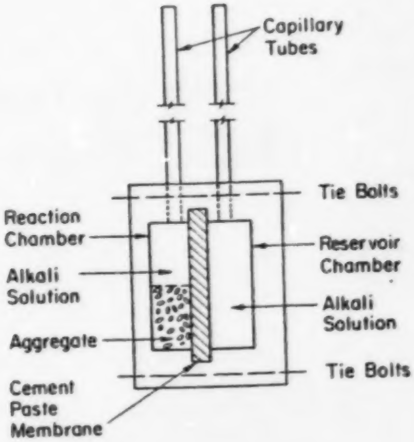


Fig. 4 - Results obtained at Laval University using the Modified Chemical Method ASTM C289 on insoluble residues for a series of carbonate aggregates from Quebec belonging to the Trenton (TR) and Black River (BR) Groups. The amounts of dissolved silica (Sc) are compared with expansion values obtained in the CSA Concrete Prism Method. All samples meeting at least one of the following criteria: (1), Sc<100 millimoles/litre; (2), Sc*<10 millimoles/litre, or (3), insoluble residue <6%, correspond to aggregates that induced concrete prism expansion <0.04% at 6 months. Using these criteria, 84% of the 19 non-expansive samples tested are classed properly (all points in the left part of the graph except the three black symbols), while all the 35 expansive samples are retained (all black symbols in the upper-right field).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Tie Bolts
Reaction Chamber
Reservoir Chamber
Alkali Solution
Alkali Solution
Aggregate
Tie Bolts
Cement Paste Membrane

Fig. 5 - Schematic diagram of the osmotic cell. From reference (42).

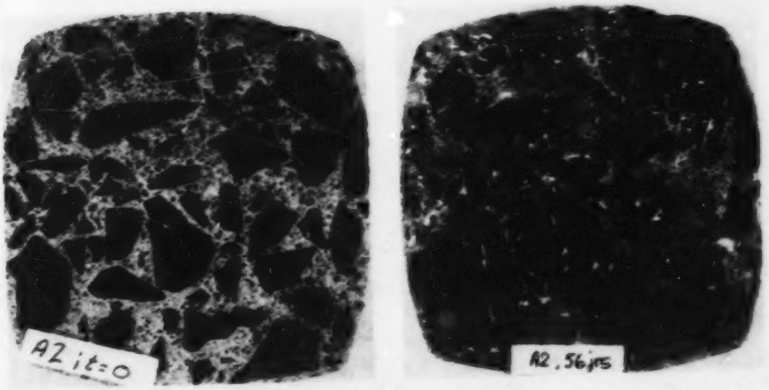
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Fig. 6 - Polished section of concrete containing reactive limestone aggregates before (A) and after (B) immersion in a 1N NaOH solution at 38°C for 8 weeks. (Accelerated Gel Pat Test). Numerous spots of siliceous gel can be seen on the aggregate particles.

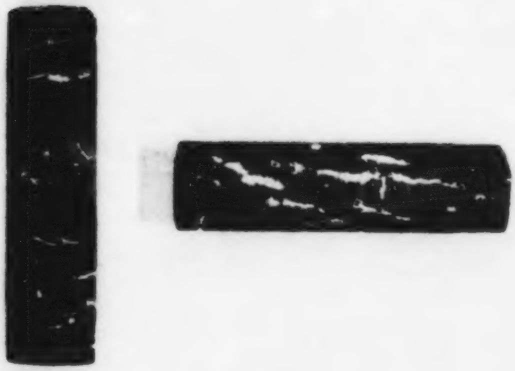


Fig. 7 - Deposits of calcium-rich alkali-silica gel on rock cylinders cored perpendicularly (at left) and parallel (at right) to the bedding of an alkali-reactive limestone from the Quebec City area (Quebec, Canada). This gel tend to be parallel to the bedding.

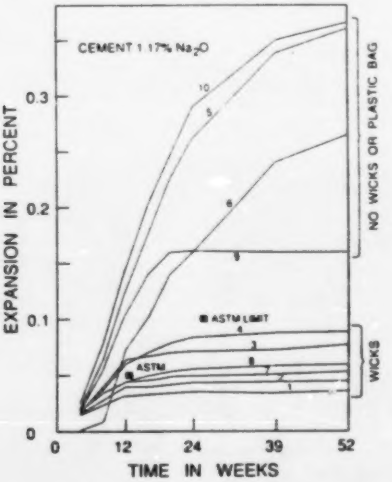


Fig. 8 - Expansion of mortar bars made in accordance with ASTM C227 with a very reactive siliceous limestone from Ottawa (Ontario, Canada), and stored in various types of containers. The presence or absence of wicks inside the containers is very critical. Container #1 is the reference container proposed in the ASTM standard. Results for natural aggregates shown on Figs. 13 and 16 were obtained using the container type #6. From reference (59).

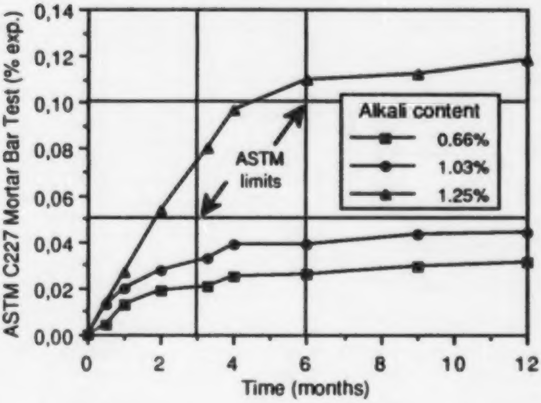


Fig. 9 - Effect of the alkali content (%Na_2O equivalent of cement mass) on the expansion of mortar bars made in accordance with ASTM C227 and using a reactive siliceous limestone from Ottawa, Ontario (Canada).

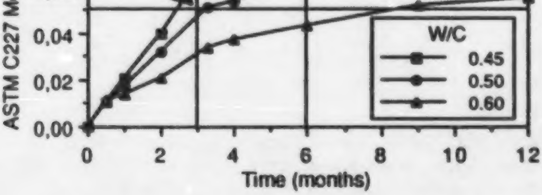


Fig. 10 - Effect of the water/cement in the ASTM C227 Mortar Bar Method. (Siliceous limestone from Trois-Rivières, Quebec, Canada).

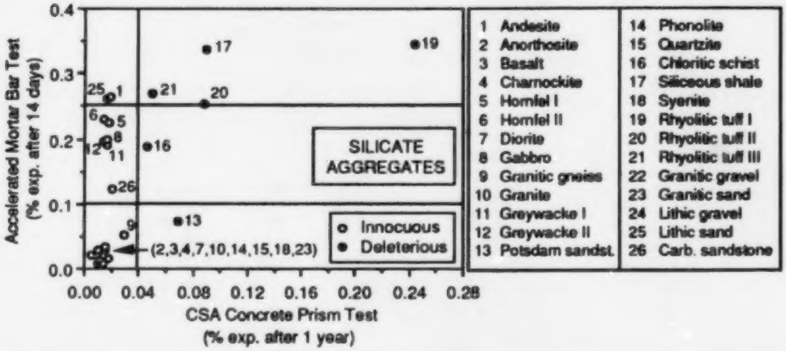


Fig. 11 - Results obtained at Laval University using the ASTM C9-Proposal-P214 Accelerated Mortar Bar Method on a series of silicate aggregates from Quebec. The results are compared to expansions obtained in the CSA Concrete Prism Method. When using a 0.10% mortar bar expansion limit, one known deleterious aggregate is classed innocuous (#13). On the other hand, a number of innocuous or presumably innocuous aggregates (#1,5,6,8,11,12,25,26) induced moderately high (between 0.1% and 0.25%), to very high (> 0.25%) expansion in the accelerated mortar bar test. In fact, only 56% of the 18 innocuous samples tested are evaluated correctly.

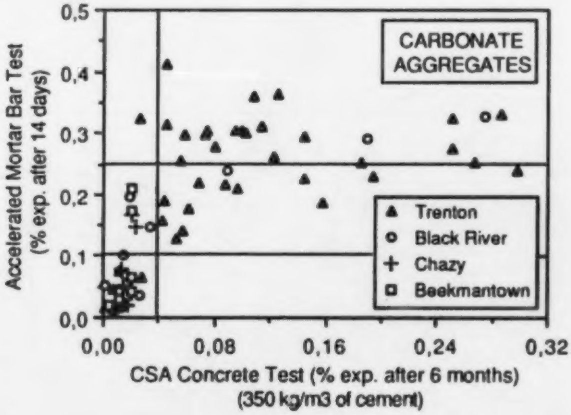


Fig. 12 - Results obtained at Laval University using the ASTM C9-Proposal-P214 Accelerated Mortar Bar Method on a series of carbonate aggregates from Quebec. The results are compared to expansions obtained in the CSA Concrete Prism Method. When using the 0.10% proposed mortar bar expansion limit, all the 35 expansive aggregates (>0.04% in the CSA concrete test) are classed reactive, while 81% of the 36 non-expansive aggregates appear innocuous (91% for the Trenton Group, 75% for Black River, 78% for Chazy, and 75% for Beekmantown).

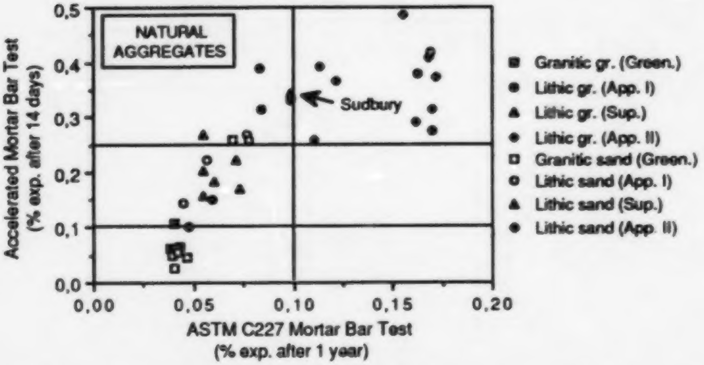


Fig. 13 - Results obtained at Laval University using the NBRI Accelerated Mortar Bar Method on a series of natural sands and gravels from Quebec. The results are compared to expansions obtained in the ASTM C227 Mortar Bar Method, using a container without wicks (type #6 on Fig. 8). Using the proposed accelerated mortar bar limits of 0.10% and 0.25% after 14 days, all expansive aggregates in ASTM C227 (exp. >0.1% after 1 year) are classed rapidly expansive (exp. >0.25%), while many innocuous or presumably innocuous aggregates appear slowly (exp. between 0.1% and 0.25%) to rapidly expansive (exp. >0.25%) in the accelerated test.

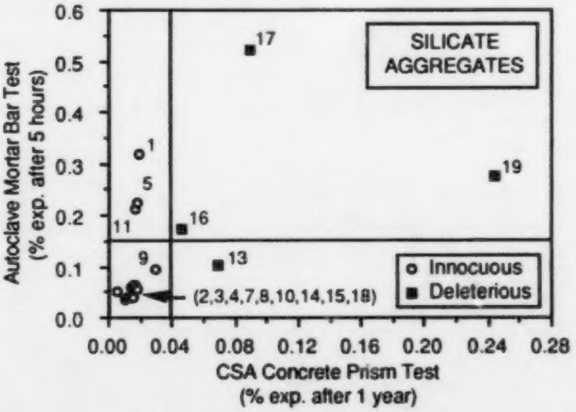


Fig. 14 - Results obtained for a series of silicate aggregates from Quebec using the autoclave mortar bar method developed at Laval University. The results are compared to expansions measured in the CSA Concrete Prism Method. When using the 0.15% proposed autoclave expansion limit, one known deleterious aggregate is classed innocuous (#13: Potsdam sandstone), while 77% of the 13 innocuous or presumably innocuous aggregates are evaluated correctly. The performance of the autoclave test is better than that of the ASTM C9-Proposal-P214 Accelerated Mortar Bar Method in which only 56% of the non-expansive samples had been evaluated correctly (Fig. 11).

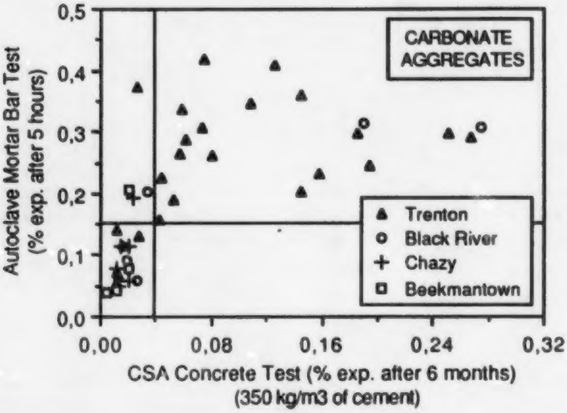


Fig. 15 - Results obtained for a series of carbonate aggregates from Quebec using the autoclave mortar bar method developed at Laval University. The results are compared to expansions measured in the CSA Concrete Prism Method. When using the 0.15% proposed autoclave expansion limit, all the 20 expansive aggregates tested (>0.04% in the CSA concrete test) are detected, while 80% of the 20 non-expansive samples are evaluated correctly. The test appears as efficient as the ASTM C9-Proposal-P214 Accelerated Mortar Bar Method (in which 81% of the non-expansive aggregates were classed innocuous; see Fig. 12).

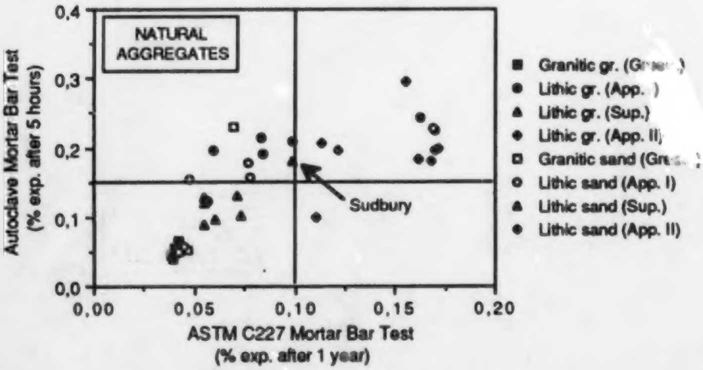


Fig. 16 - Results obtained for a series of natural sands and gravels from Quebec using the autoclave mortar bar method developed at Laval University. The results are compared to expansions measured in the ASTM C227 Mortar Bar Method, using a container without wicks (type #6 on Fig. 8). When using the 0.15% proposed autoclave expansion limit, one expansive sand in ASTM C227 (expansion >0.1% after 1 year) is classed innocuous, while many innocuous or presumably innocuous aggregates appear reactive in the autoclave test.

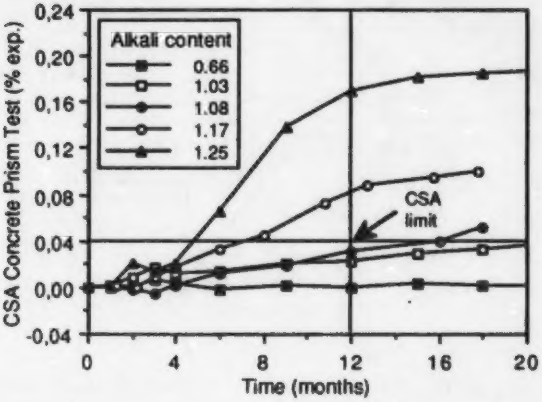


Fig. 17 - Effect of the alkali content (%Na_2O eq. of cement mass) in the CSA Concrete Prism Method. (Rhyolitic tuff from Quebec, Canada).

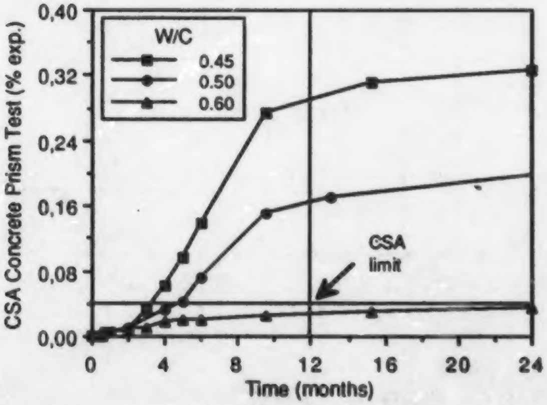


Fig. 18 - Effect of the water/cement in the CSA Concrete Prism Method. (Siliceous limestone from Trois-Rivières, Quebec, Canada).

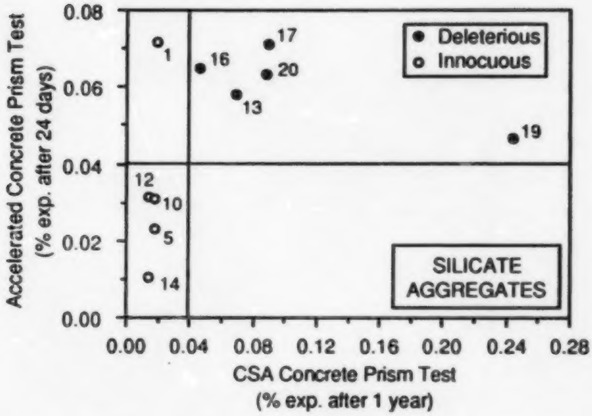


Fig. 19 - Results obtained at Laval University using the Accelerated Concrete Prism Method (immersion in NaOH 1N at 80°C) on a series of silicate aggregates from Quebec. When using the proposed limit of 0.04% expansion after 24 days, the 5 known deleterious aggregates tested are classed reactive, as well as one of the 5 presumably innocuous aggregates tested (#1: andesite). (Refer to Fig. 11 for aggregate identification).

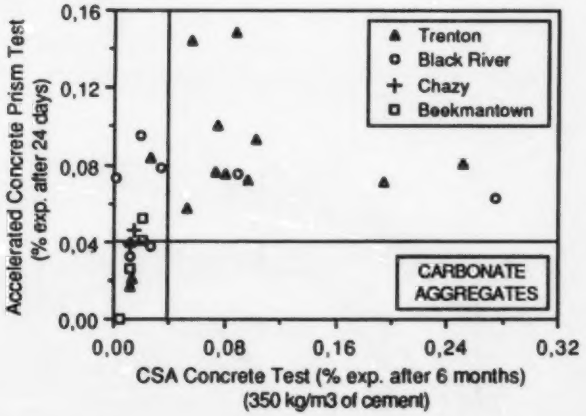
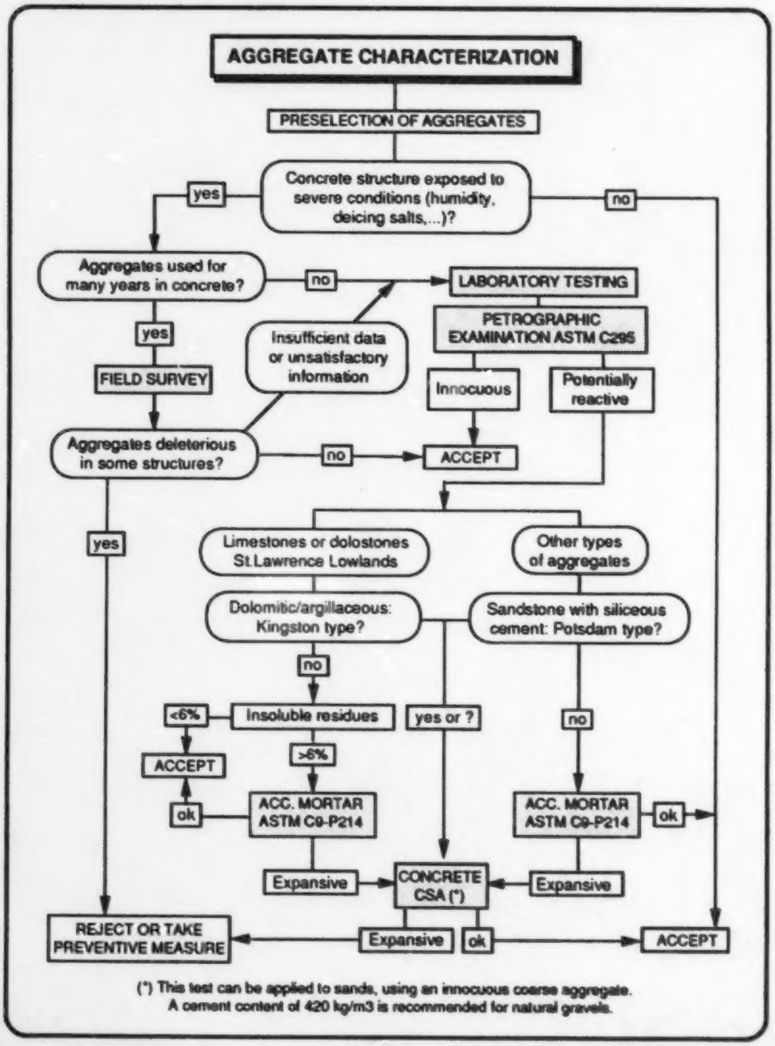


Fig. 20 - Results obtained at Laval University using the Accelerated Concrete Prism Method (immersion in NaOH 1N at 80°C) on a series of carbonate aggregates from Quebec. When using the proposed limit of 0.04% expansion after 24 days, all the 12 expansive aggregates tested (>0.04% in the CSA concrete test) are classed reactive, while only 53% of the 15 non-expansive aggregates are evaluated correctly. Then, the performance of this test is worse than that of the ASTM C9-Proposal-P214 Accelerated Mortar Bar Method in which 81% of the innocuous aggregates tested had been classed properly; see Fig. 12).


AGGREGATE CHARACTERIZATION
PRESELECTION OF AGGREGATES
Concrete structure exposed to severe conditions (humidity, deicing salts,...)?
yes
no
Aggregates used for many years in concrete?
no
LABORATORY TESTING
yes
PETROGRAPHIC EXAMINATION ASTM C295
Insufficient data or unsatisfactory information
FIELD SURVEY
Innocuous
Potentially reactive
Aggregates deleterious in some structures?
no
ACCEPT
yes
Limestones or dolostones St.Lawrence Lowlands
Other types of aggregates
Dolomitic/argillaceous: Kingston type?
Sandstone with siliceous cement: Potsdam type?
no
<6%
Insoluble residues
yes or ?
no
ACCEPT
>6%
ok
ACC. MORTAR ASTM C9-P214
ACC. MORTAR ASTM C9-P214
ok
Expansive
CONCRETE CSA (*)
Expansive
REJECT OR TAKE PREVENTIVE MEASURE
Expansive
ok
ACCEPT

(*) This test can be applied to sands, using an innocuous coarse aggregate.
A cement content of 420 kg/m3 is recommended for natural gravels.

Fig. 21 - Decision chart proposed for determining the potential alkali-reactivity of concrete aggregates. This chart is based mainly on experience in Quebec (Canada).

Fibre-Reinforced Cement and Concrete

by Colin D. Johnston

Synopsis: Differences and inconsistencies in the terminology applied to fibre-reinforced cementitious matrices in various countries are clarified. Concepts governing the interaction of fibres of any type with a cementitious matrix are identified. Problems and limitations associated specifically with the incorporation of fibres into cement paste, mortar, or concrete are discussed in terms of maximum possible fibre volume fraction, fibre form, geometry, orientation, distribution, and compatibility with the manufacturing process. Methods of characterizing the behavior of fibre-reinforced cements and concretes are examined in terms of their suitability for demonstrating the effects of fibres with respect to strengthening, toughening, and resistance to crack development. The history of fibre development and use in pastes, mortars, and concretes is reviewed for glass, steel, polypropylene, carbon and other fibre groups such as aramids, acrylics, polyesters, and natural cellulose-based fibres. The nature and degree of property improvements is cited for each fibre type in terms of strength, toughness, and crack development under static loading in tension, compression, or flexure, and, when possible, for dynamic impact or fatigue loading and restrained shrinkage conditions that induce cracking. Circumstances where incompatibility of fibre and matrix may lead to long-term deterioration in properties are identified. The scope of past and present applications is summarized for each fibre type.

Keywords: Fibres, Cement Paste, Mortar, Concrete, Proportioning, Workability, Strength, Toughness, Shrinkage, Cracking, Durability.

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INTRODUCTION

The idea of reinforcing relatively brittle building materials with fibres has been known and practised since ancient times. Mud huts constructed from baked clay reinforced with straw, and masonry mortar reinforced with animal hair, are primitive examples of the use of fibre-reinforced materials in construction that date well back into history. The first modern and more sophisticated example of a fibre-reinforced construction material mass-produced to a predictable level of quality is asbestos-cement sheeting manufactured by the Hatschek process from about 1900.

The basic principle in both historic and recent examples is to enhance the properties of an inherently weak and brittle cementitious matrix by adding fibres to increase tensile strength and ameliorate brittleness giving improved ductility and toughness. This principle remains largely unchanged today, but the diversity in fibre types, shapes, and sizes and in chemical and mineral admixtures available to enhance matrix characteristics has broadened greatly in recent years.

Definitions

The wide range of composites made possible by combining various fibres with cement paste, mortar, or concrete has led to some inconsistency in terminology as well as different spelling of the primary keyword. In its first state-of-the-art report on the subject in 1973(1) the American Concrete Institute defined *fiber-reinforced concrete* as "concrete made of hydraulic cements containing fine, or fine and coarse aggregate, and discontinuous discrete fibers". It endorsed this definition again in 1982(1). In 1977 its counterpart in Europe, RILEM(2), defined *fibre concrete* as "made from hydraulic cements with or without aggregates of various sizes and incorporating, in the main, discrete fibre reinforcements".

Neither the term *fibre concrete* nor the term *fiber-reinforced concrete* distinguishes adequately between matrices consisting solely of cement paste, those containing fine aggregate and cement paste (mortar), and those containing fine and coarse aggregate with cement paste (concrete). Commercial product terminology in everyday use can be similarly misleading. For example, while the term asbestos-cement implies a blend of asbestos fibres, cement and water, it sometimes also contains, finely divided silica, sand and other additives(3,4). Likewise, the material known as glass fibre-reinforced cconcrete (GFRC) in the U.S.A. or glass-reinforced cement (GRC) in Europe may contain fine sand in addition to cement, water, and cement replacements such as fly ash or slag(3), in which case it is technically a mortar. The same misleading terminology is found in many technical papers purporting to discuss the properties of fibre-reinforced concrete, which on careful reading reveal that coarse aggregate was not included. Instead, the materials discussed are in fact fibre-reinforced mortars or pastes that often have properties far superior to their equivalents in concrete because the volume of fibres that can be accommodated in a cementitious mixture increases with increase in the volume fraction of the paste.

Scope

Since the first edition of this chapter was written in 1978, an abundance of new information has been published on all aspects of the subject, some of which were essentially unexplored in 1978. While this edition attempts to be as broad and comprehensive as possible, it cannot quote all relevant references because the list would occupy too high a proportion of the available space. Priority in listing references is therefore given mainly to recent publications that are readily accessible, so that the interested reader can pursue any aspect of the subject further using the reference lists in these recent publications. Major compilations with papers by different authors on many aspects of the subject are identified separately.

FIBRE-MATRIX INTERACTION

Fibres in a cementitious matrix (paste, mortar, or concrete) can in general have two important effects. First, they tend to reinforce the composite under all modes of loading which induce tensile stress, eg. direct tension, bending, and shear, and, secondly, they improve the ductility and toughness of an otherwise inherently brittle matrix. The improvements in these properties imparted by any particular fibre depend on many fibre parameters, notably the fibre content which is usually identified as percent by volume of composite, V_f, and also on the mode of loading. For example, in compression the strengthening effect is often small while the toughening effect is usually substantial.

Direct Tensile Loading

Direct tension is the mode of loading best suited for conceptually understanding the composite action of fibres and matrix because the stress distribution is uniform and the fibre-reinforcing effect is primarily in this mode. The understanding can then be extended to other modes of loading that are more important in practical applications, such as bending and shear, where the normal or principal stresses are in part tensile(5,6,7).

Stress-strain behavior in direct tension(5) can be idealized in stages as in Fig. 1 starting with the matrix carrying most of the load and the fibres carrying relatively little since their volume is quite small compared with that of the matrix (OA in Fig. 1)(5). As microcracks develop into a continuous crack over the whole loaded area, load is progressively transferred by shear at the fibre-matrix interface until it is carried entirely by the fibres (A in Fig. 1). If the fibres are short and their volume fraction is small, they may be unable to sustain the load transferred, and either break because their tensile strength is exceeded, or, under conditions of increasing strain, pull out of the matrix with a progressive decrease in composite stress because of breakdown of the interfacial fibre-matrix bond (AB in Fig. 1). On the other hand, if the combination of fibre volume fraction and length is sufficient, the fibres may sustain the total load while cracking of the matrix progresses at nearly constant stress (AC in Fig. 1). Subsequently, the composite stress may increase further as the available strength or pullout resistance of the fibres develops to a maximum (CD in Fig. 1). In reality, the idealized two-stage behavior (ACD in Fig. 1) is often continuous without a distinguishable break at C, and in the extreme may reach AE for composites like asbestos-cement with high strength and comparatively low toughness as represented by area under the stress-strain curve. What happens in any particular case depends on the fibre volume fraction and the following intrinsic fibre characteristics(4):

(i) Tensile Strength -- The fibre must be substantially stronger than the matrix, since the effective load-bearing area of a typical concentration of fibres (1-5% by volume) is much less than that of the matrix (99-95% of the total volume). However, the tensile strength of cementitious matrices is so low that this is no problem for most fibres (Table 1).

(ii) Elongation at Failure -- The fibre must be able to withstand strains well in excess of the strain at which the matrix cracks. Again, cementitious matrices are so brittle that this is not usually a problem (Table 1).

(iii)Elastic Modulus -- The higher the elastic modulus, the greater is the proportion of load carried by the fibres in a loaded composite prior to cracking, provided that the fibres and matrix remain fully bonded. Also, the higher the modulus the less the strain in the composite after cracking, so modulus is important when comparing the merits of different fibres (Table 1).

(iv) Interfacial Bond Shear Strength -- Transfer of load from the matrix to fibres is contingent upon shear bond at the interface between them (Fig. 2)(8). Interfacial bond involves varying degrees of adhesion, friction, and mechanical interlock (similar in effect to deformed rebar). Unfortunately, the factors governing bond of a particular type of fibre to a particular cementitious matrix are numerous(9-14), and, even when bond strength is established in single-fibre uniaxial pullout tests, it does not correlate well with the measurable mechanical properties of the composite in which the fibres are randomly oriented and may interact with each other(11).

(v) Poisson's Ratio -- Differences in Poisson's ratio between fibres and matrix (0.20 - 0.25 for most pastes, mortars or concretes) create stress across the fibre-matrix interface transverse to the fibre. If Poisson's ratio of the fibres is greater than that of the matrix, this transverse stress is tensile and the fibre tends to tear away from the matrix adversely affecting adhesional and frictional bond (Fig. 2)(8). If Poisson's ratio of the fibre is less than that of the matrix, the stress is compressive and the frictional component of bond is increased.

(vi) Elasticity -- If the fibre is not truly elastic, and instead is subject to creep under load, it suffers stress relaxation in a loaded composite, and the degree of strengthening imparted decreases with time. The performance of such fibres can therefore depend on rate of loading. Many polymeric fibres exhibit significant creep at normal temperatures (Table 1). At elevated temperatures, creep increases and reductions in both strength and elastic modulus may become substantial even for fibres like steel and glass.

Certain physical and geometrical characteristics of the fibre also have a major effect on its effectiveness:

(i) Aspect Ratio -- This is the ratio of length to diameter for fibres of circular cross section. (An equivalent diameter $d_e = \sqrt{4A/\pi}$ is used for non-circular cross sections of area A). Although the fibre and interfacial shear stress distributions are in general non-linear (Fig. 2), the effect of fibre aspect ratio can be illustrated using simple linear stress distributions (Fig. 3) to show that when the composite is loaded failure may occur in one of two ways. If the fibre is shorter than a certain critical length, ℓ_c, the fibre stress σ_1 is less than the fibre strength σ_{max} and the fibre pulls out of the matrix due to interfacial bond failure (Fig. 3 left). If it is longer than ℓ_c, the fibre eventually fails in tension because the fibre stress equals its tensile strength (Fig. 3 right). The reason is that in the range $o<\ell<\ell_c$, equilibrium of the forces due to interfacial shear strength, τ_{max}, and tensile stress in the fibre, σ_1, dictates that $\sigma_1 = \tau_{max}\ \ell/d$ (6). Therefore, the maximum stress attainable in the fibre, and thus its reinforcing effectiveness, are related to the aspect ratio of the fibre.

Generally, composite strength increases with increasing fibre aspect ratio for ℓ/d values less than ℓ_c/d (Fig. 3 centre). While there is in theory a critical aspect ratio, ℓ_c/d at which fibres may be regarded as operating at peak efficiency, mixing considerations often render this apparent ideal unattainable in practice. The reason is that fibres having the required aspect ratio clump together in balls that are difficult to disperse before and during the conventional concrete mixing process. Consequently, a major consideration in developing fibres suitable for conventional concrete mixers has been to improve resistance to fibre pullout without increasing fibre length to the point where aspect ratio becomes high enough to cause fibre balling.

(ii) Shape and Surface Texture -- Obviously, any technique for increasing the interfacial shear bond strength increases the stress attainable in the fibre, and improves its effectiveness. Such techniques include physical processing to produce fibres with deformed or roughened surfaces, enlarged or hooked ends, or a lengthwise profile which is crimped or wavy rather than straight. All three methods have reached the production stage for steel fibres (Fig. 4).

(iii) Absolute Size -- There is some evidence(15) indicating that, regardless of aspect ratio, the fibre length must exceed the maximum aggregate size in the matrix. Thus, very small fibres which are long enough to be effective in cement paste and mortar may not be as effective in concrete as larger longer fibres of the same aspect ratio.

(iv) Form -- Some types of fibre are produced in monofilament, multifilament or fibrillated (network) form. For example, while smooth plastic monofilaments like polypropylene adhere poorly to cement paste, fibrillated tapes with a branched network structure offer improved mechanical bond(16,17) (Fig. 5). Likewise, multifilament glass fibre strands bond better with cement paste than monofilaments because of improved mechanical interlock (Fig. 5).

(v) Surface coatings -- Coating with substances intended to improve the adhesional component of bond(10,18) or to protect the parent fibre material from damage during mixing and chemical attack by the cement constituents(19,20,21), may also influence fibre effectiveness.

For straight round fibres, improvements in strength, ductility and toughness imparted by any particular fibre largely reflect the combined influence of fibre volume and aspect ratio(22). Enlarging the ends of fibres or modifying the surface texture or form to increase pullout resistance have the same effect as increasing the aspect ratio, but avoid increasing the fibre length and consequently making mixing and uniform fibre distribution in the matrix unduly difficult.

As an upper bound extreme, it is useful to consider composites with high volumes of very long fibres, fibres that are continuous and aligned (like concrete reinforcement) in the example shown for glass fibres in Fig. 6(5). Strength improvements depicted by the maximum stress compared with the probable tensile strength of the unreinforced cement paste (3-5MPa) are up to 2000%. Likewise, ductility is many times the probable cracking strain of the unreinforced matrix ($100\text{-}500\times10^{-6}$). Toughness represented by total area under the stress-strain curve also increases dramatically. Such composites demonstrate the importance of fibre content, but are outside the scope of the definitions given earlier for fibre-reinforced concrete, mortar, or paste. Nevertheless, the progression from them to composites with high volume fractions of discrete randomly oriented fibres (Fig. 7) is logical because large improvements in strength, ductility and toughness are again associated with increasing fibre content(14). Fibre aspect ratio, as represented by length for the multifailament glass fibres in Fig. 8, is also important(14).

While fiber volumes of 2-10% are possible for composites with paste or mortar matrices prepared by special blending or infiltration processes, conventionally mixed concrete can only accommodate 0.5-2.0% volume of fibres, and property improvements in direct tension(22) are accordingly less. Nevertheless, the combined importance of fibre volume and aspect ratio, as expressed by their product VL/D, is still obvious(22) (Fig. 9). For fibres with ends modified to improve pullout resistance, toughness is improved compared with straight uniform fibres(23) (Fig. 10).

Other Modes of Loading

Compression -- Cracking of the matrix is not produced directly by compressive stress, so fibers have a negligible effect in the direction of applied stress. However, the standard compression test under uniaxial loading produces a combination of shear failure near the ends of the specimen with lateral swelling and cracking of the central portion of the specimen when the lateral strain exceeds the cracking strain in tension, so fibres can have some reinforcing effect in the lateral direction. For the 1-2% fibre volume typical of conventionally mixed fibre concretes their effect on strength is generally minimal, but strain capacity and consequently toughness as represented by area under the stress-strain curve are often significantly improved. The magnitude of these improvements depends on fibre type and on the product of fibre volume and aspect ratio, VL/D(23,24) (Fig. 11). At the higher fibre volumes (12%) possible in paste-reinforced composites where the fibres are aligned to some degree, the fiber effect is similar but depends also on the direction of fiber alignment relative to the axis of loading(25).

Flexure -- Composite behavior in flexure reflects the combined influence of factors already identified for tension and compression. However, unlike the parent matrix in which the distribution of stress is essentially elastic (Fig. 12a), the stress distribution in most fibre-reinforced cementitious composites is far from elastic in the tension zone(6).

An analytically simple approximation based on a plastic distribution of stress in the tension zone(5,6,7) that is supported by experimental data for steel fibre-reinforced pastes, mortar and concretes(26) explains why fibres are much more effective in flexure than in direct tension (Fig. 12b). In this configuration, if f_t is able to maintain a value σ_t after cracking has occurred, the moment of resistance for fibre-reinforced concrete (Fig. 12b) is 2.4 times the corresponding moment based on simple elastic analysis (Fig. 12a). If f_t remains greater than 0.41 σ_t after cracking, M_2 exceeds M_1. This implies that if the post-crack tensile strength of the composite at large strains is at least 0.41 times the maximum stress reached in direct tension, flexural strengthening can occur up to a flexural load capacity corresponding to a stress of 2.4 times the tensile strength based on elastic analysis (Fig. 12a).

While the simple plastic distribution of stress is useful in explaining why fibres are more effective in flexure than in tension, the actual stress distribution may vary depending on fibre volume and type(6,7) (Fig. 13).

Shear -- In considering shear, a distinction must be made between composites where the fibre orientation is 2-dimensional random in the plane of the section, as in thin-section spray-up glass fibre-reinforced cement, and others in which the fibre orientation is truly 3-dimensional random, as in thick-section fibre-reinforced concrete beams or slabs.

In composites with 2-d fibre orientation, shear resistance on planes parallel to the plane of the section, termed interlaminar shear, is not significantly improved by the fibres, so the shear strength and ductility correspond to the properties of the parent unreinforced matrix. Consequently, failure by delamination is possible. Shear properties normal to the plane of the section are much greater, and shear strength is approximately equal to the direct tensile strength(27,28). For composites with 3-d random fibre orientations, like conventionally mixed thick-section concretes, where shearing through the matrix alone without involving the fibres is very unlikely, shear strength increases attributable to fibres for torsional loading(26) and for discs in punching shear(29) are about the same as for direct tension (Fig. 9), so there is again evidence of equality between shear strength and tensile strength.

As many as seven other forms of shear test based on cubes, cylinders or prisms notched in a variety of ways to induce shear failure have been attempted(30), and strength increases are insignificant to modest, 40% for composites with fiber volumes of less than 1.5%. They can be much greater in high fibre volume composites like glass fibre-reinforced cement with 5-10% of fibres(27,28,30). Toughness can also improve substantially even at lower fibre volumes(30).

Effect of Fibre Orientation and Distribution

The orientation of a fibre relative to the plane of a crack strongly influences its ability to transmit load across the crack. A fibre which happens to parallel the crack has no effect, while one perpendicular to the crack has maximum effect. The overall reinforcing effectiveness of fibres in a matrix depends on the number intersecting a unit area(31), and on the pullout resistance as determined by factors such as aspect ratio, shape and surface texture.

In mass concrete, fibre orientation is normally 3d-random, and the reinforcing effectiveness is the same in all directions. In thin sheet-like products, and to some extent in shotcrete, the fibre orientation is essentially 2d-random, and the reinforcing effectiveness is primarily in directions parallel to the plane of the section. Although strength and toughness in these directions are considerably higher than for 3d-random orientation, strength perpendicular to the plane necessarily suffers causing relatively poor resistance to failure by delamination.

Consolidation by vibration can induce a degree of preferential 2d-fibre orientation parallel to the horizontal plane in conventionally mixed and placed fibre-reinforced concretes (32,33). This may be beneficial in cases where the preferential orientation coincides with the direction of the major tensile stresses, eg. the horizontal directions in a pavement. Aligned 1d-fibre orientations can be produced by special techniques(34,35,36) (Fig. 14), but are of limited practical significance as conventional reinforcement is more effective in this role.

Formwork or mold surfaces cause some fibre alignment parallel to the contact surface, and the effect is most significant for small test specimens where the fibre length is large relative to the minimum crosssectional dimension of the test specimen. Therefore, small molded specimens can exhibit property improvements higher than the mass concrete they purport to represent(37) (Fig. 15). This is in addition to the effects of specimen size and loading configuration on the flexural strength of the unreinforced matrix(38) (Fig. 16)

Concentrating the fibres in regions of maximum stress rather than distributing them uniformly throughout the structural element is not only more efficient in a technical sense (Fig. 14 left), but offers the prospect of significant cost savings where the use of fibre-reinforced concrete is partial rather than total. For example, it has been shown that the flexural strength of a beam reinforced with fibres only in the bottom sixth of the depth is comparable to that of a beam with the same volume of fibres uniformly distributed(34,39).

Effect of the Composite Manufacturing Process

To be effective as reinforcement fibres must not be significantly damaged by the process used to incorporate them into the matrix. Processes available vary considerably in terms of the severity of bending, impact, and abrasion inflicted on the fibres.

Least severe is the spray-up process whereby chopped fibres and separately mixed cementitious slurry are sprayed onto a molding surface, usually from separate nozzles, so no mechanical mixing of the fibres with the matrix is required and the possibility of damage to fibres is minimal. There are many variations on this process that can include automation of the spraying operation, dewatering of the freshly prepared composite, subsequent folding and forming, and the option to omit fibres in some of a series of sprayed layers(14). Then, there are various premix processes based on preparation of a cementitious slurry followed by the blending of fibres into it in the mixer afterwards, thus minimizing the time the fibres are exposed to mixing and possible damage. Segregation, water loss, poor fibre dispersion and non-uniform fibre distribution can be prevented, by various additives(14). Type of mixer is also important. Pan mixers are preferable to drum mixers, and current practice for glass fibre-reinforced cement is to prepare the matrix by high shear mixing and then transfer it to a paddle type mixer prior to adding fibres to minimize mixing time with the fibres present(14).

Processes developed historically for asbestos cement have been adapted to suit other types of fibre(3,14). Most important is the widely used Hatschek process that involves preparation of a mixture of slurry and fibres in continuously agitated dilute suspension followed by pick up of a thin layer of the suspension on a rotating cylinder of fine wire mesh and transfer of it to a conveyor belt of permeable felt with vacuum dewatering.

Most severe are the various conventional concrete mixing processes where rotation of the mixer, the shearing action of the blades, and contact with aggregate particles, subjects the fibres to a high degree of bending, impact and abrasion. Some fibres such as glass are quite unsuitable for such processes as they sustain breakage causing loss of aspect ratio and reinforcing effectiveness(40). Multifilament strands like glass also tend to separate with a consequent increase in surface area that adversely affects workability. A high energy flexible base type of mixer has been developed specifically for use with carbon fibres to achieve better fibre distribution and less fibre breakage(41). Fibres in the form of fibrillated tapes like polypropylene are intended to separate or defibrillate during mixing to improve the mechanical bond provided by the secondary branches, but overmixing can cause fibre damage and reduce effectiveness.

Shotcreting like conventional mixing involves consideration of fibre damage by bending, impact, and possibly to a lesser extent abrasion. Both dry-mix and wet-mix processes have been widely accepted for fibres resistant to these forms of damage, primarily steel and to a lesser extent polypropylene(42).

A process termed SIFCON(43) that is especially suited to incorporating much higher fibre volume fractions (5-18%) than are possible with conventional mixing or shotcreting involves preplacement of the fibres in a mold followed by infiltration with a slurry of paste or mortar. Although used mainly with steel fibres to date, the avoidance of mixing makes it potentially suitable for other fibers at high volume concentrations primarily in thin sections, especially if it can be automated to reduce labor costs.

Compatibility of Fibre and Matrix

The physical and chemical compatibility of fibres with the moist alkaline environment characteristic of cement paste is important. In the short term, fibres which absorb significant amounts of water may cause excessively rapid loss of workability in concrete, in much the same way as dry-batched lightweight aggregate. In the long term, fibres may be attacked by water or the alkalis present in solution in cement paste. For example, many cellulose-based vegetable fibres such as cotton, sisal and hemp rot in a moist environment, and the common forms of glass fibre based on E-glass deteriorate rapidly on contact with the dissolved alkalis present in hydrated cement paste. Fibre-specific compatibility problems are discussed subsequently for each fibre type.

CHARACTERIZING COMPOSITE PERFORMANCE

Cementitious matrices are very brittle with cracking strains in direct tension in the range $100\text{-}500\times10^{-6}$ depending on the water-cement ratio of the paste and the presence or absence of aggregate, $400\text{-}500\times10^{-6}$ for low water-cement ratio pastes (5,14,44) down to as little as 100×10^{-6} for concrete with 20 mm aggregate(45). Therefore, the composite strain when the matrix first cracks, point A in Fig. 1, is expected to equal the cracking strain of the matrix in tension, ϵ_m. The composite stress, σ_c, at this point is accordingly the product of this strain and a composite modulus $E_c = \eta E_f V_f + E_m V_m$, where V and E denote volume proportions and moduli respectively of the matrix and fibres, and the coefficient η accounts for discontinuous randomly oriented fibres. This coefficient is unity for continuous aligned fibres, but its value for discontinuous fibres is less than 1.0 and depends on many factors such as the degree of alignment, if any, fibre aspect ratio, and interfacial bond strength. However, even when it is assumed as 1.0, increases in σ_c compared with matrix strength σ_m have been shown by experimental data to exceed expectations. For example, increases in strength with 1.84% volume of steel fibres in mortar can exceed 25% compared with the 10% predicted from the above equation assuming $\eta = 1.0$ and $E_f/E_m = 6$(Fig. 9). Increases in strength for higher fibre volumes of glass fibres also exceed the prediction of approximately 25% for 8.2% fibres with $E_f/E_m = 4$ (Fig. 7). This is attributed to the notion, confirmed by some experimental observations, for example in Fig. 9, that the composite cracking strain exceeds the matrix cracking strain, and much has been published to take account of this in theory(5,14).

What happens in terms of post-crack behavior beyond A in Fig. 1 is even more difficult to explain theoretically (46,47,48,49) because the concepts of stress and strain uniformly distributed throughout the material become largely invalid, and only load and deformation are readily determinable.

Nevertheless, from a practical viewpoint two important effects of fibres on composite behavior can be distinguished.

Strengthening

Regardless of whether the composite is a low-fibre-volume concrete or a high-fibre-volume paste, there are levels of stress and strain at which it first exhibits cracks that are significant in continuity, width, visibility and total length. First crack is the point on the stress-strain or load-deformation relationship at which it first deviates clearly from linearity indicating a decrease in composite stiffness caused by the onset of cracking. This point is variously referred to as first crack, the proportional elastic limit (PEL), limit of proportionality (LOP) or bend-over point (BOP), and it has been shown to correspond to the maximum acoustic event rate in acoustic emission measurements to monitor crack development(50). Below this point, even though there is progressive microcrack development, the matrix contributes to strength, but at this point cracking has become visible and continuous enough to decrease stiffness significantly and render the matrix contribution negligible.

For most composites first crack is readily distinguishable in tension and flexure (Fig. 7,10), although for some highly reinforced ones the decrease in stiffness is slight and the first-crack point more difficult to define accurately (Fig. 6). In compression, first crack is not clearly identifiable because of the curvilinearity of the stress-strain relationship (Fig. 11).

Since the first-crack strength characterizes composite behavior up to the onset of significant and usually visible cracking in the matrix, it has obvious practical importance in relation to the maximum permissible design load under uniquely defined serviceability conditions where cracking to any extent is not permissible. On the other hand, the maximum or ultimate load, and any strength derived from it, may correspond to the first-crack equivalent (curve OAB in Fig. 1) or may exceed it (curve OAD in Fig. 1), and accordingly may correspond to conditions of deformation/deflection and cracking much more severe than at first crack.

In flexure in particular, the ultimate load has little significance, other than in relation to maximum permissible short-term load, because it cannot be associated with a definite serviceability condition in terms of deflection and cracking (Fig. 17a). In contrast, first-crack strength is related to a definite serviceability condition and is a meaningful strength parameter because the assumption of elastic bending used to derive it is valid up to cracking. This assumption is no longer valid once cracking occurs, and a modulus of rupture or flexural strength based on maximum load is accordingly meaningless.

Toughening

Cement paste is extremely brittle, and, although fine and coarse aggregates modify this characteristic slightly in mortar and concrete by hindering crack propagation, both materials are still very brittle when compared with other common construction materials like steel or wood. Fibres that are strong and ductile can effectively inhibit the rapid and unlimited propagation of cracks associated with catastrophic failure in the unreinforced matrix (Fig. 17b). Their presence causes slower and more widely distributed cracking that develops gradually with increase in load. Their bridging action also enables transfer of stress across cracks, allowing the affected part of the composite to retain some residual post-crack strength and to withstand deformations much larger than can be sustained by the unreinforced matrix alone (Fig. 17a).

In general, under slow loading conditions, often termed static loading, no matter what the mode of loading (tension, compression, flexure etc.), the result of both inhibited more widely distributed cracking and crack bridging by fibres in the composite is a substantial increase in the areas under the stress-strain or load-deflection curves (Fig. 7,11,17a). The property characterized by these areas is commonly referred to as toughness, and represents the work of fracture or energy absorption capability of the composite. The relevance of toughness in practice depends on the application. In the extreme, where the resistance of a structural element to total collapse is important, as in earthquake or explosive loading, the whole area under the stress-strain or load-deformation curves may be the most relevant measure of toughness. More frequently, where retention of a given level of serviceability under design conditions is required, only a portion of the total area under the curves is likely to be important. In such cases, toughness is more appropriately based on an area defined in accordance with serviceability conditions expressed in terms of a limiting deflection, deformation, crack width or residual composite strength. Under serviceability conditions that do not permit significant cracking, for example bridge decks exposed to deicing salts, the appropriate area may be only the area up to first crack.

Improvements in toughness are strongly influenced by fibre content (Fig. 7) and the resistance of the fibres to pullout, which is governed primarily by fibre aspect ratio in the case of uniform straight fibres (Fig. 8,9) and by the other features of shape, surface texture, lengthwise profile or end configuration for non-uniform fibres, is also very important (Fig. 10).

Under impact loading which in laboratory tests is produced by falling-weight or weighted-pendulum arrangements that usually cause complete fracture of the test specimen, the work of fracture or impact resistance is also improved by fibres(51-55). However, an additional and important consideration is that fibres which induce only small improvements in toughness under normal static loading because of an apparently low modulus can offer significantly improved impact resistance under high-rate loading because the effect of time-dependent viscoelastic behavior is reduced or eliminated. For example, polypropylene fibres are reported(16) to exhibit an elastic modulus under impact loading of two to three times the normal value for static loading.

In principle, single-cycle impact tests with appropriate instrumentation(51,52,53) produce stress-strain or load-deformation curves that can characterize toughness in the same way as static loading tests. On the other hand, multiple-cycle empirical impact tests without sophisticated instrumentation, such as the drop-weight procedure(54,55), cannot characterize toughness in a fundamental way.

Under explosive loading, which is basically a form of impact loading, fibres are particularly beneficial in reducing both the amount of fragmentation and the fragment velocities(56,57). However, these measures of toughness are not easily related to the work of fracture assessed in static or impact tests.

More complex still is behavior under repeated cyclic loading which produces fatigue failure. While improvements in fatigue strength attributable to fibres have been documented for some steel and polypropylene fibre concretes(58-60), the development of parameters to define toughness under fatigue loading has received little attention. One form of definition has been proposed for compression(61) that involves integrating the stress-strain functions for each successive cycle of load to obtain a total area representing all cycles of load applied to the prescribed end-point strain.

Strengthening Versus Toughening

Nowadays, there are many techniques for increasing matrix strength using chemical and mineral admixtures, but only fibres significantly increase toughness. Therefore, in characterizing the performance of fibre-reinforced cementitious composites under load, it is useful to distinguish between strengthening and toughening.

Certain fibres when present in sufficient amounts may substantially increase both the load or stress at which cracking first occurs and the ultimate load or derived stress(13,62) (Fig. 7 and 18a). Many others produce small or negligible increases in the first-crack strength, but even in small amounts cause some significant increase in the area of the load-deformation curve after cracking(48) (Fig. 18b). This represents toughness in the composite that is almost entirely attributable to the fibres because it is absent in the unreinforced brittle matrix (Fig. 18a), except under special closed-loop strain-controlled testing arrangements which depict a post-crack portion of the stress-strain curve for the unreinforced matrix (Fig. 18b) not obtainable under normal testing conditions (Fig. 18a).

For some composites, the first-crack strength may be quite small and the toughness very large (curve c in Fig. 19), while for others the first-crack strength may be quite high and the post-crack toughness very small or negligible (curve e in Fig. 19). What happens in any particular case depends entirely on the factors that affect fiber-matrix interaction both in the short and long term, including exposure conditions (Fig. 19). However, from a design viewpoint it is important not only to distinguish between strength up to first crack and toughness after cracking, but to identify the true nature of the composite behavior after cracking. This means distinguishing between strain-softening, perfectly plastic, and strain-hardening behaviors after cracking (Fig. 17a,20a) because engineers relate readily to the relatively safe nature of elastic-plastic behavior typical of mild steel, are comfortable with strain-hardening behavior provided the strain capacity is large, and regard severe strain-softening after first crack as undesirable for many applications.

Toughness represented by some portion of the area under the load-deformation or stress-strain curve does not necessarily distinguish the strain-hardening or strain-softening tendencies of composites with different first-crack strengths (Fig. 20a), or the associated differences between them with respect to residual strength at large deformations(Fig. 20b). For example, two composites may have the same toughness in terms of the area under the load-deflection curve in flexure and be quite different with respect to first-crack strength and residual strength after cracking(63) (Fig. 20b).

Toughness defined in terms of area under the load-deformation curve has the disadvantage of being specimen-specific, that is numerically very dependent on specimen and testing variables, so it is not in any sense universally comparable for the range of specimen sizes and testing arrangements encountered in practice. What is needed are parameters that define composite behavior after first crack in a universally applicable and readily understandable way regardless of specimen and testing variables.

In principle, any toughness index calculated by dividing the area under the load-deformation curve up to a specified end-point deformation by the area under the same curve up to first crack is independent of specimen size and testing variables. Furthermore, when the end-point deformation (strain in tension or deflection in flexure) is chosen as an appropriate multiple of the first-crack deformation, a scale of subscripted toughness indices evolves that indicates the average slope of the post-crack portion of the curve (Fig. 21a). This definition system, termed the base-unity system because all toughness indices have a value of 1.0 for the unreinforced brittle matrix, provides an effective means of comparing the performance of different composites with a readily understandable reference, elastic-plastic ($I_5 = 5.0$, $I_{10} = 10.0$ etc.), as illustrated in principle (Fig. 22).

An alternative definition system that is the same in principle is the base-zero system in which all indices have a value of zero for the unreinforced matrix (Fig. 21b). It differs from the base-unity system in that the numerator used to calculate the toughness index is the area between the first-crack deflection and the specified end-point deflection, that is the area up to first crack is excluded instead of included.

The importance of any system that allows the end-point deformation to vary (n in Fig. 21) is that it can be applied to a wide range of anticipated serviceability conditions defined in terms of permissible deformation or crack width. An end-point corresponding to the index I_5 may be appropriate for an application where only minimal cracking is permissible, while an end-point corresponding to I_{50} or I_{100} may be justifiable for an application where resistance to complete structural disintegration under, for example, earthquake loading is the ultimate concern.

The base-unity system became the basis of ASTM C1018(66) in 1984 for flexural testing of fibre-reinforced concrete (Fig. 23). It is applicable in principle to direct tension as well as flexure for concretes where the composite behavior up to first crack is essentially linear and the change in stiffness (slope of the curve) at first crack is significant and easily distinguishable. For compression, where the composite behavior is quite curvilinear, some modification is necessary, such as basing toughness indices on the point at which the maximum load is first reached and on the triangular area OAB rather than the actual area under the curve up to A (Fig. 24). This modification ensures that index values have much the same meaning as for flexure and tension, that is index values equal to their subscript correspond to plastic behavior after the maximum load. An alternative in which the base deflection δ corresponds to 85% of the maximum load in compression has been proposed(67), but in this system index values are not easily related to a readily understandable reference such as plastic behavior after maximum load.

While the toughness indices I_5, I_{10}, I_{20} etc.(65,66) indicate the overall degree of strain-hardening or strain-softening after first crack, the shape of the post-crack portion of the load-deformation curve may depart significantly from the uniform slope depicted in the simple schematics of Fig. 22, as illustrated by curves for real composites (Fig. 20b,25). In terms of the strength retained at any deflection after first crack, curves A and B (Fig. 20b) differ significantly, and to many engineers the concept of retained strength is more easily understood than an index defining strain-hardening or softening. Accordingly, the originally proposed concept for toughness indices(65) was extended in 1986(68) to include the determination of residual strength factors (Fig. 23). This concept was incorporated into ASTM C1018(66) in 1989.

In flexure, since stress at any deflection beyond first crack is not determinable because the relationship between load and stress is no longer governed by the elastic bending formula, it is logical to express the retained load capacity as a percentage of the first-crack load and term it a residual strength factor without reference to units of load or stress because the first-crack strength is already identified in either of these units. Direct tension is simpler because a post-crack stress is determinable. For both modes of loading, the concept leads to an easily understood scale of residual strength factors where elastic-plastic behavior corresponds to 100 and brittle behavior to 0. Residual strength factors may be established for specific deformation/deflection values or expressed as an average over a specified interval. In the latter case, they are readily derived from toughness indices I_5, I_{10}, I_{20} etc. for intervals corresponding to the end-points for these indices (Fig. 23).

An example of how the toughness indices and residual strength factors determined according to ASTM C1018(66) fully define composite behavior in terms of the strengthening and toughening imparted by fibres in flexure is presented for four steel fibre-reinforced concretes that differ significantly in fibre type, fibre content, and matrix strength (Fig. 25)(63). Toughness indices close to the reference elastic-plastic values of 5, 10 and 20 correspond to essentially elastic-plastic or yield-like behavior (curve 1 in Fig. 25). Lesser indices denote strain-softening behavior with similar values indicating about the same degree of softening regardless of first-crack strength (curves 2 and 4 in Fig. 25). Nearly constant residual strength factors denote an essentially constant proportion of the first-crack strength retained to the maximum deflection interval considered (curves 1 and 3 in Fig. 25). Residual strength factors decreasing progressively as deflection increases denote significant strain-softening (curves 2 and 4 in Fig. 25).

Another proposed system for defining toughness indices(69-72) also recognizes the principle that defining the test end-point as a multiple of the first-crack deformation/ deflection makes the resulting toughness indices conceptually independent of specimen size, shape, and mode of loading. After several stages of development Fig. 26a(69,70,71), what evolved was essentially a base-zero system on a percentage scale having an index value of 0 for the unreinforced matrix and 100 for a composite with linear elastic behavior to failure (Fig. 26b). Unfortunately, index values for the elastic-plastic behavior level more typical of many composites vary with end-point deflection. However, the results again illustrate the expected influence of fibre volume on composite behavior (Fig. 27), and are interesting because different types of specimens and modes of loading have been evaluated, including standard and notched flexural specimens(71), indirect tension specimens(72), and several configurations producing shear(30). Notching was found to have a very small effect on toughness indices in flexure(70,71) (Fig. 28), and indices for a form of indirectly applied tension(72) were much less than for flexure.

The main attribute of both the systems of defining toughness and residual strength just discussed is that they are applicable in principle to any shape or size of specimen and any flexural, direct tensile, and with some modification, compressive loading configuration. Even though the actual values of the parameters measured may not be entirely independent of specimen and testing variables, just as the commonly used measure of the quality of plain concrete, compressive strength, is not independent of these variables, their meaning as measures of composite performance and quality is universal regardless of specimen and testing variables.

In contrast definition systems that use arbitrarily chosen deflection or load values as the test end-point instead of an end-point defined as a multiple of the first-crack deflection, such as those endorsed by ACI Committee 544(54), the Japan Concrete Institute/Japan Society of Civil Engineers(73) and RILEM Committee 49TFR(74) give values of toughness and toughness indices that are specimen-specific. They have no applicability to other specimen shapes, sizes or modes of loading, and no relationship to a reference level of composite behavior such as elastic-plastic. Furthermore, none of these systems provides for selecting the test end-point to match anticipated serviceability conditions.

Numerous other definition systems dealing with toughness continue to be proposed(24,67,75,76,77)., but none of them have been widely accepted to date, and they are not discussed further.

Performance in flexure has generated the most interest, but experience(64) has shown that accurate measurement of deflection is essential, usually by means that eliminate the influence of deflections at the beam supports(64,73). Under these conditions the ASTM C1018(66) procedure is very effective in distinguishing differences in the performance of fibre-reinforced concretes attributable to fibre type and amount(78).

Crack Development

In addition to strengthening and toughening, a third important facet of composite performance is the manner in which fibres inhibit or modify crack development. In general, any form of loading that produces tensile stress causes a degree of cracking that increases with the magnitude of the load. It can be monitored in terms such as maximum crack width, mean crack width, total width for all cracks, and mean crack spacing. In flexure, the progressive increase in total crack width with increase in load correlates quite well with ASTM C1018(66) toughness indices(63) (Fig. 29), which is consistent with the idea previously expressed that the end-point for toughness testing can be selected to match anticipated serviceability conditions in terms of cracking.

Crack development can also occur in the absence of any form of direct loading in applications where shrinkage of the cementitious matrix is sufficiently restrained to cause tensile stress exceeding the tensile strength, so the effect of fibres on crack development has been extensively studied in restrained shrinkage tests. Most investigators (79-83) have employed a steel ring or cylindrical core around which the fibre-reinforced composite is cast (Fig. 30a). After moist curing the test specimens are dried, and they shrink against the restraining ring or core. Heat can be applied to the ring or core to accelerate the test. Drying may occur through all three exposed surfaces or may be restricted to the outer surface by applying sealant to the others. In all cases the principle is that cracks develop progressively in number and width as drying progresses. Their number, width, location and other characteristics can be monitored, optionally in conjunction with free shrinkage measurements on standard prisms. as, for example, in Fig. 30b(79,80).

Instrumentation of the steel restraining ring with strain gauges and use of the measured compressive strain in the steel to calculate the tensile stress in the concrete using elastic theory is also possible(81). In general, the fibrous composites are able to sustain much greater stresses due to restrained shrinkage before cracking than the unreinforced matrix, but the drying time to produce cracking depends strongly on fibre type and amount(81) (Fig. 31).

A major problem with monitoring crack development is detection and measurement of the width of very small cracks, so refinements incorporating laser holography for detection, and high power microscopes (100× with a precision of 2.5 μm) for accurate width measurement, have facilitated study of the effects of fibres on cracking(82), as has the development of an explanatory analytical model(83). One of the more interesting conclusions(82) is that in addition to the expected influence of fibre volume on cracking (Fig. 32), the effectiveness of small percentages of fibres is much greater for steel than for polypropylene, probably because of differences in modulus and interfacial fibre-matrix bond strength (Fig. 33). Another is that the free shrinkage of ring specimens without restraint corresponds closely to the value determined on standard prisms.

While ring tests are useful for comparing the performance of composites with different types and amounts of fibres under a specific drying regime, applying the results to establish joint spacing criteria in the field, where specimen size and drying conditions differ greatly from the test is at present an unsolved problem. With regard to size, most ring specimens are only 35-75mm thick(79-83) because it is too difficult to provide restraint sufficient for cracking in thicker ones, so preferential fibre alignment parallel to the ring circumference is unavoidable. Therefore, ring specimens tend to perform better than thick structural elements where fibre alignment is less pronounced, especially with fibres longer than twice the specimen thickness or depth.

Alternatives to the ring specimen(82) that require further study are slabs restrained at the edges(84) or with a centrally placed restraining insert(85). Neither has yet been widely accepted for evaluating the effects of fibres on cracking caused by long-term drying shrinkage. Instead, the emphasis in these types of test has been on studying the effects of fibres on plastic shrinkage cracking during the first few hours after placement (84,85).

FIBRE DEVELOPMENT AND USE

The history of fibre development and use in cementitious matrices dates from the late 1800's when asbestos-cement was first produced. Glass fibres have been available since about 1920, but attempts to utilize them effectively in cement and mortar, which evolved largely from the successful development of fibreglass, started in the 1960's. Patents covering the use of short pieces of steel in concrete date from 1913 onwards(86), but their present-day use in concretes stems largely from work performed after 1964. Interest in using other types of manufactured fibres in cementitious matrices expanded in the 1970's and 1980's largely as a result of their development and successful use in other fields. They include carbon, polypropylene, polyethylene, polyester, acrylic, aramid, and certain natural fibres such as coconut coir, sisal, jute, and wood pulp (Table 1).

The use of fibres in cement paste differs significantly from their use in concrete. First, the manufacture of cement-based products such as asbestos-cement and glass-reinforced cement (GRC) involves blending the fibres with cement slurry without exposing them to the rigour of a conventional concrete mixing process, so compatibility of the fibres with the manufacturing process is much less critical than for fibre-reinforced concrete. Second, the products are usually thin sheet, so the fibres necessarily adopt a 2d-random orientation instead of the primarily 3d-random orientation characteristic of typical concrete products, and property improvements for equivalent fibre volumes are therefore greater than for fibre-reinforced concrete. Third, since fibres occupy the cement paste fraction of a mortar or concrete matrix, the maximum fibre volume possible in terms of maintaining reasonable mixture consistency is much greater in pastes and mortars than in concretes(26) (Fig. 34), and property improvements are therefore less in the concretes.

Effect of Fibres on Freshly Mixed Composites

Adding any type of fibre progressively reduces mixture consistency. This has proven to be more critical for conventionally mixed fibre-reinforced concretes than for cement-based composites fabricated in other ways, like asbestos-cement and GRC. Nevertheless, the use of additives, sometimes with vacuum dewatering, is common in the production of nearly all such composites to achieve mixture consistency sufficient for fabrication without excessively high water-cement ratio in the finished product.

For conventionally mixed concrete loss of workability increases with the volume concentration and aspect ratio of the fibres regardless of type(26,87) (Fig. 35, 36). Before the advent of superplasticizers, the disadvantages associated with the extra water otherwise needed to maintain workability could eventually outweigh the benefits imparted by the fibres, but this is less of a problem nowadays with widespread acceptance and use of superplasticizers. However, the apparent effect of fibres is strongly dependent on the method of measuring workability(87,88). Judged on the basis of slump alone, the workability may appear too low for placement because fibres impart considerable stability to the slump sample(87). If workability is judged by V-B time, which is a measure of the ability of the concrete to flow under vibration, it remains adequate over the quite wide range of fibre contents where the V-B time is 3 to 9 seconds, even though the corresponding slump is quite low(87) (Fig. 35). Since the V-B apparatus, although excellent for laboratory use, lacks the portability normally desirable in the field, the inverted slump cone test was proposed as an alternative for field work(54). It involves measuring the time taken for the inverted slump cone to empty under internal vibration into a standard yield bucket, and became ASTM Standard C995 in 1983(89). Times of 8 to 30 seconds are indicative of concrete suitable for placement by vibration(87)(Fig. 35).

The linear relationship(87) (Fig. 37) between V-B and inverted cone times suggests that each test measures essentially the same mixture characteristic, namely mobility or ability to flow under vibration, which is more important in terms of present-day placement practices than the stability or cohesion measured in the slump test.

Glass Fibres

Following the successful development and use of borosilicate E-glass fibres in glass-reinforced plastics in the 1950's, their potential for use in concrete was explored, but it was quickly realized that such fibres deteriorate rapidly in the alkaline environment of cement paste. Subsequently, considerable work was directed towards developing an alkali-resistant glass fibre. A zirconia-based glass developed by Corning Glass Works in the U.S.A. was fiberised into multifilament strands (204 filaments per strand) that led to the production of Cem-FIL fibres in the U.K. in the 1970's. The technical and production details that led to wide commercial use of alkali-resistant Cem-FIL fibres and subsequent attempts to further improve the resistance to alkali attack by additives and coatings are readily available(14). For example, the inclusion of a chemical inhibitor in the coating applied to glass fibres during manufacture that is slowly released into the cement paste surrounding the fibres to reduce glass-cement interaction, led to the development of Cem-FIL 2 fibres(14) (Fig. 38).

Glass fibres are inherently brittle and quite prone to damage or strand separation by impact and abrasion during conventional concrete mixing processes, so their use in concrete has been very limited(90,91,92). Strands of length 13-52mm added to conventionally mixed concrete (10 mm aggregate) improved first-crack and ultimate flexural strengths in accordance with fibre volume concentration(90) (Fig. 39). Glass strands reinforcement in mortars (5 mm aggregate) have been shown to improve flexural strength(91) and slightly reduce shrinkage and creep(92).

Use of glass fibre strands in cement paste matrices where the production process involves spraying of chopped strand and cement slurry separately onto a molding surface, thus avoiding the potentially damaging conventional concrete mixer, has been widespread(14). The thin sheet-like products of glass-reinforced cement, often termed GRC or GFRC, can accommodate high fibre volumes in a two-dimensionally aligned orientation. Short-term properties depend on strand length and concentration, as shown for direct tension (Fig. 7 and 8). Flexural performance is probably more relevant for most applications, with strand length and amount again most important(14) (Fig. 40 and 41). However, because of the possibility of glass-cement interaction adversely affecting long-term properties under moist exposure conditions, short-term properties are less of a concern than their long-term equivalents.

Long-term natural weathering effects and accelerated tests to simulate them over a shorter time span have been performed for properties such as flexural strength, impact resistance, and toughness (area under the load-deflection curve to failure) (93-96). The initial thrust has been to establish the severity of deterioration and the time span over which it occurs(94) (Fig. 42). Eventually, it seems that composite properties approach values for the unreinforced matrix(96) (Fig. 43, 44), so emphasis has been on reducing the rate of deterioration and prolonging the time span.

The alternatives examined include coatings(19) (Fig. 45), the addition of silica fume to suppress the alkali-silica reaction(20), Cem-FIL2(14) (Fig. 46), and the addition of polymer additives to the matrix with a reduction in mixing water(14,97). Eliminating normal cement with its alkalis and lime as a hydration product, and replacing it with high-alumina cement or supersulphated cement have also been quite successful (Fig. 19) (14,93). A new low-alkali Japanese cement, termed CGC, that is claimed to produce no lime during hydration appears promising with alkali-resistant AR fibres, but not with the much cheaper E-glass fibres(19,98) (Fig. 47). Other attempts to utilize the cheaper E-glass fibres using an acrylic polymer emulsion with normal cement have not been successful in totally eliminating property deterioration with age under moist exposure conditions(13,96,99).

Durability continues to be perhaps the most important issue governing the use of GRC in a diversity of environments that can differ substantially in terms of temperature and the presence of moisture. Compilation J(19,20,21,99) was devoted entirely to this subject in 1986.

Applications -- The early applications explored by the Building Research Establishment (BRE) in the U.K. included cladding panels, pipes, window frames and even permanent formwork(3). Because of reductions in strength, toughness and impact resistance with time in moist environments GRC must not be used in a primarily load-bearing structural capacity. BRE Digest 331 (100) presents guidelines and lists prohibitions.

Currently, the largest category of applications is in wall systems variously identified as cladding panels, architectural facades, sandwich panels that incorporate thermal insulation, prefabricated steel-stud frames integral with a GRC skin (1,14,27,101,102). ASTM C947 and C948(103) deal with this category of applications, and recommended practices and tests have also been developed by the Prestressed Concrete Institute(27), and RILEM(74). The tests deal with flexural strength and toughness, absorption, permeability and impact resistance.

The second largest application category is in surface-bonded masonry where concrete blocks, normally dry stacked with mortar only used to the minimum necessary to keep them plumb, are coated on both vertical faces with a layer of GRC mortar applied by trowelling or spraying. This greatly improves the flexural and overturning resistance of block walls which, although strong in compression, are weak in flexure. ASTM standards C887 and C946 have been developed for this application(103).

Other applications include sewer pipes, water pipes and channels, formwork, and agricultural, mining, and marine uses too numerous to describe in detail(1,14,27,104).

Steel Fibres

Initially, round steel fibres were produced by cutting smooth high tensile steel wire of uniform crossection, much of it rejected portions of the wire used in steel-belted radial tires. To improve the quite low resistance to fibre pullout of smooth uniform fibres bonded solely by adhesion to the cement paste, a surface-deformed version was developed and patented under the name Duoform. The objective was to augment the adhesive bond by introducing mechanical interlock between fibre and matrix in the same way that deformed rebar improves bond compared with smooth rebar. Fibres prepared relatively cheaply by cutting thin sheets of steel, often referred to as slit-sheet, were also initially available with a uniform rectangular crossection. Subsequently, both the wire and slit-sheet fibres appeared with enlarged or hooked ends to improve pullout resistance, and fibres crimped to a continuously wavy profile over their entire length, again to improve pullout resistance, became available. More recently, fibres have been produced in a variety of crimped or straight surface-roughened forms with or without enlarged ends by machining directly from billet steel, thus avoiding the expense of intermediate processing to wire or plate.

Further developments have produced fibres cast directly from molten steel, often referred to as melt-extract, again with the possibility of enlarged ends or roughened surfaces to improve pullout resistance. However, perhaps the most important feature of melt-extract fibres is the ability to incorporate alloys into the melt to alter the inherent properties of the steel, notably its resistance to corrosion, by including small amounts of chromium and other alloying elements. These fibres are corrosion-resistant without the full complement of alloying elements needed for the more expensive stainless steel. The microstructure and properties of the steel are influenced by the melt temperature and rate of cooling, extremely rapid cooling being used in at least one process to produce an amorphous glass-like structure(105).

Carbon-steel fibres are available in a wide variety of forms, shapes and sizes shown in Fig. 4. Corrosion-resistant alloyed steel and fully stainless steel fibres are available in melt-extract form for purposes where their extra cost can be justified by requirements for corrosion resistance or high temperature performance in refractory composites.

Steel fibres are used mainly in concrete rather than in mortar or cement paste, where, unlike many other fibres, they are not adversely affected by conventional mixing. Some of the developments already mentioned for improving pullout resistance also assist in maximizing fibre performance in hardened concrete without incurring undue loss of workability during mixing. Instead of increasing aspect ratio by increasing fibre length and thus adversely affecting workability in the freshly mixed state (Fig. 36), enlarging or hooking the fibre ends, surface texturing, and crimping, all permit the fibres to be physically shorter and more easily mixed without the loss of pullout resistance that shortness would otherwise cause. Fabricating fibres in bundles held together with a water-soluble glue (Fig. 4) facilitates dispersion of long fibres that would individually cause mixing and balling problems. Steel fibres for concrete are dealt with in ASTM A820(106). Recommended practices for mixture proportioning, mixing, placing and finishing of steel fibre-reinforced concrete have been developed(40).

The volume of steel fibres used in concrete is less than is possible in mortars or pastes for the reasons previously discussed (Fig. 34) with maxima usually of 0.75-1.5% (60-120 kg/m^3). Improvements in direct tensile properties are modest (Fig. 9), but of limited practical interest, while improvements in flexure are more substantial because of the unique stress distribution produced by fibres (Fig. 12). Although initially promoted by some fibre manufacturers on the basis of greatly increasing composite strength in flexure, steel fibres impart only modest increases in ultimate strength(22) and in first-crack strength(68,107). Compared with plain concrete, increasing the first-crack flexural strength by more than 40% would be exceptional for fiber amounts up to 1.5% by volume(68)

(Fig. 48). In contrast, increasing the I_{10} toughness index tenfold (plain concrete has $I_{10} = 1.0$ by definition) to achieve elastic-plastic composite behavior is quite possible(68) (Fig. 49). With the major improvement in toughness comes substantial improvement in residual strength. The residual strength factor $R_{5,10}$ can increase from 0 for the matrix to the 100 corresponding to elastic-plastic behavior, or more, for some type-amount fibre combinations (Fig. 49). Generally, the improvements in toughness indices and residual strength factors are highly dependent on fibre amount, aspect ratio, and type (Fig. 25,49) and largely independent of matrix parameters (Fig. 50). On the other hand, first-crack strength is highly dependent on factors that govern the matrix strength such as water-cement ratio, silica fume etc. (Fig. 50).

Under impact loading conditions, wide-ranging claims of improvements attributed to fibres have been reported mostly using the empirical drop-weight test(54,55,107) where the actual energy input to the specimen is impossible to determine. However, in fully instrumented flexural impact tests using the weighted pendulum (Charpy) technique(51), it has been demonstrated that increases in impact resistance correspond closely to increases in toughness measured under slow (static) flexural loading(64) (Fig. 51).

There have also been wide-ranging claims of improvements in the flexural strength under repeated cyclic fatigue loading(58,59). Much depends on how the improvements are presented. Sometimes they are stated relative to a concrete without fibres(59), which is usually not equivalent in all respects to the matrix of the fibre-reinforced concrete. Sometimes they are related to the first-crack or ultimate strengths under static loading(59,60) (Fig. 52). Both the latter include whatever improvements the fibres impart to the static strength of the matrix (Fig. 48). Since flexural fatigue failure follows almost immediately after the appearance of cracking, the improvements imparted by fibres in fatigue, as distinct from static loading, are probably best illustrated in terms of performance relative to the first-crack strength under static loading. When presented in this manner they show the expected effects of fiber content, aspect ratio, and type (Fig. 52). What is not yet entirely clear is whether steel fibre-reinforced concrete, unlike plain concrete, exhibits a true endurance limit beyond which it can sustain a certain level of stress indefinitely(59,60).

Steel fibres like conventional reinforcement are completely compatible with the alkaline environment that normally prevails within cement paste. Only when the alkali is leached out or when the paste carbonates does the pH drop sufficiently for fibre corrosion to occur in uncracked concrete. Even under conditions of marine exposure allowing penetration of chlorides, corrosion has not been observed beyond the surfacial carbonated zone(108, 109,110). The splash zone that combines wetting and drying cycles and sometimes freezing and thawing cycles is more critical than the fully submerged zone(109,110). Under conditions involving freezing and thawing, steel fibres do not significantly enhance performance(111). A proper air-void system is essential, coupled with the normal maximum limit on water-cement ratio appropriate to the exposure condition(109, 111,112). Steel fibres do however appear to moderate the degree of volume increase and loss of dynamic modulus associated with the cracking caused by freezing, especially in concretes with inadequate entrained air(112) (Fig. 53). However, they are not a substitute for a good air-void system.

Cracks in steel fibre-reinforced concrete render the steel fibre directly vulnerable to whatever corrosive agents are present in the surrounding environment. Experiments on precracked specimens in a moist marine atmosphere(113) showed that corrosion over a 570-day period was not serious at crack widths less than 0.15mm. Similar experiments under more severe conditions involving circulated calcium chloride solution, showed severe corrosion at crack widths as low as 0.05mm(114). For carbon-steel fibres there is probably no absolutely safe maximum crack width(115), and the corrosion rate will depend on fibre type, crack width, and the severity of the exposure condition. However, since the maximum fibre stress is usually much less than the fibre strength, steel fibres may continue to carry load across cracks even when their crossectional area is considerably reduced by corrosion. It follows that large diameter fibres may be better than smaller ones in the long term. Nevertheless, use of alloyed fibres that are corrosion-resistant or corrosion-proof may ultimately be the only way to effectively address corrosion of steel fibres in cracked concrete(106,115).

Applications -- The early applications up to 1975 included pavement construction and pavement overlays for airfields, highways, bridges and industrial floors, maintenance repairs to dams subject to severe cavitation, shotcrete lining in mining, tunneling, slope stabilization and a variety of repairs to bridges, culverts, etc.(116). Pavement overlay work and shotcrete applications comprised the largest proportion of the total in the early 1980's(117), and ACI Committee 544 complied a list of 112 projects worldwide that was made available from ACI since 1981(118). The material has also found a wide variety of uses in precast elements as diverse as dolosses to control ocean wave action, secured safes and vaults, tilt-up panels, single family automobile garages, and mine crib blocks(1).

Flatwork -- The largest volume usage over the last decade is in flatwork, and an extensive listing that includes paving, overlay, and industrial floor projects has been compiled with information on design, mixture proportions and performance(119). Generally, the objectives have been to achieve performance equivalent to plain or reinforced concrete with reduced thickness and increased joint spacings.

Problems that have been encountered in pavements and overlays include concerns about exposed or loose fibres at the surface, particularly with regard to ingestion by jet engines, corner or edge curling and related cracking, and load transfer at joints(120). There seems to be little documented evidence that the exposed or loose fibre problem has ever actually had serious consequences, but it has led to greater emphasis on placement and finishing(121). The curling problem can be addressed by avoiding the high cement contents and water contents that exacerbated shrinkage and heat of hydration in many early projects, and by using fly ash, water-reducing admixtures, larger aggregate etc. There is some debate about whether joint spacings should be decreased or increased and whether thickness should be increased to improve flexural stiffness(120). Small joint spacings reduce curling, but are undesirable for smooth passage of vehicles and are uneconomic. Increasing thickness and stiffness by compositing the more expensive fibre-reinforced concrete with a cheaper underlying concrete(120,122) may permit larger joint spacings without excessive curling. Related to the curling problem is the omission in many of the early designs of load transfer devices such as keyways and dowels because the pavements were considered too thin to accommodate them(120,123). The longer the joint spacing the more critical is joint design with respect to the provision of keyways, dowels and tie bars. There is some evidence that dowels and tie bars create more problems than they solve, and that properly designed keyways or partial-depth sawn joints are the best way of addressing the curling and load transfer problems for large slabs. While there is not complete agreement on what constitutes optimum joint design, experience suggests maximum spacings of 12m for transverse joints and 8m for longitudinal joints(120,123).

For flatwork applications where the fluctuations in temperature and humidity that cause curling are small, notably industrial floors that are enclosed within a building soon after construction, much longer joint spacings are justifiable. Many large projects have been completed, at first mainly in Europe(124), and more recently in N. America(125). This application category includes not only new floors but thin overlays to restore existing floors. A warehouse floor comprising four 30x40m bays free of joints has recently been constructed using laser-controlled screeds (126).

Overlays introduce the particular problem of bond with the underlying pavement. Fully bonded overlays are less prone to curling but more prone to cracking caused by shrinkage or thermal change, unless the joints are properly located. Reflection cracking occurs unless joints are matched with joints or working cracks in the underlying layer(119). Debonding achieved by placement on double polyethylene sheeting or a bituminous leveling course has been quite successful in many applications(119), but thin unbonded overlays may be more prone to curling. There appears to be no general agreement on whether overlays should be fully bonded, or unbonded. Thin overlays should probably be fully bonded but thicker ones may be better unbonded.

Shotcrete -- Steel fibres have been used in both dry-mix and wet-mix shotcrete in a wide variety of applications involving both new construction and repair(127,128). Steel fibres comprise by far the largest volume usage of fibres in shotcrete(128). Steel fibres make possible the omission of the traditional mesh reinforcement resulting in significant cost savings and thickness reductions along with improved performance(129,130). Slope stabilization, tunnel linings, or linings for other natural or manufactured containment devices, including underground storage facilities and above-ground domes or similar thin-section structures, are among the applications for new construction using both dry and wet processes, in some cases with robot-controlled placement techniques (128,130,131, 132). Steel fibrous shotcrete has also been used extensively in repairs to tunnels, bridges, culverts, buildings and marine structures damaged by cracking, salt attack, freezing and thawing and other causes(128,132,133).

Silica fume with steel fibres has been found to produce a superior shotcrete that combines the advantages of each with respect to strength, toughness, and impermeability, ease of application and problems with adhesion, cohesion and rebound(133,134).

Structural Precast -- The miscellany of potential applications cannot be fully described here, but the interested reader will encounter the majority of them in collected publications on fibre-reinforced concrete such as the Dec. 1984 and March 1985 issues of ACI Concrete International and Compilations G,H,I, and K identified at the end of this chapter. Uses in structural connections subjected to severe loading such as seismic shock, and replacement of stirrup reinforcement for shear in beams, have received considerable study, but neither has been widely exploited yet in applications.

Polypropylene Fibres

Initially, polypropylene fibres were available as smooth monofilaments of uniform crossection with the low modulus and poor adhesion to cement paste characteristic of the parent material. To supplement the low adhesional bond with a degree of interlock or mechanical bond, a process called fibrillation became widely used to create a branched network structure. This involves extrusion from a die producing a tubular or flat film which is split into tapes and stretched over rollers with pin systems that produce longitudinal splits and a mesh-like network after stretching(135) (Fig. 5). Alternatively, the film may be split by rubbing to produce more random lengths of longitudinal splits. In another variation of the process, the splitting produces a fibre crossection that is irregular with frayed edges resulting in discontinuous secondary filaments attached to the continuous mesh(17,136), again to promote better mechanical bond. Yet another approach to improving bond and pullout resistance is to create surface roughness with mineral particles embedded in the fibre surface that subsequently achieve a hydraulic bond with cement hydration products(137). Improving the mechanical bond by branching or surface processing and the elastic modulus by stretching continue to receive considerable attention(138).

Polypropylene is attractive as fibre reinforcement because of its excellent resistance to moisture, acids or alkalis, and the cheapness of the raw material on a volume basis compared to steel or glass(138). It is used in two ways, one involving continuous aligned mesh-like reinforcing layers in a paste or mortar matrix(135,136), and the second involving discontinuous randomly oriented short fibres in conventionally mixed concrete or shotcrete(139).

In the first case the volume of the fibres in the continuous reinforcing layers can approach 10% because conventional mixing is avoided. Property improvements are substantial, often with composite behavior well above elastic-plastic (Fig. 54) and energy absorption capability superior to asbestos-cement and glass fibre-reinforced cement(136) (Fig. 55).

In the second case, the volume of fibres is much smaller because of their extremely high surface area that makes it difficult to incorporate more than 0.75% by volume of long fibres (40mm or more) into conventionally mixed concrete with 20mm coarse aggregate (Fig. 56). Higher volumes up to 2% are possible with shorter fibres (less than 20mm) and small aggregate(17). Improvements in toughness indices and residual strength factors (Fig. 23) depend on fibre content and length(133,139) (Fig. 57). Resistance to shrinkage cracking under conditions of restraint is also improved(82) (Fig. 32,33). Durability in cement under conditions of natural and accelerated weathering is excellent(140,141). Predictions from alkali immersion tests at temperatures up to 90°C indicate that polypropylene will retain 90% of its strength for at least 50 years at 20°C(142). However, both fibre and composite strength decrease sharply at temperatures exceeding 100°C that approach the melting point of polypropylene(142).

Applications -- A high proportion of the applications involving continuously aligned fibres in cement paste or mortar mixtures have aimed at replacing the traditional thin-sheet applications of asbestos-cement, notably corrugated sheeting(135,143,144, 145). Corrugated sheeting has also been produced with some types of discontinuous fibrillated polypropylene fibres along with a variety of thin-sheet precast products for buildings, water tanks and garden furniture(17,146).

Applications that involve discontinuous fibres in concrete utilize as little as 0.1% fibres by volume in flatwork to reduce or eliminate the possibility of plastic shrinkage cracking within the first few hours after placement. Residential flatwork where the degree of supervision and quality control is often inferior to what prevails on larger jobs has been prominent(147). However, while small amounts of polypropylene fibres may reduce plastic shrinkage and plastic shrinkage cracking(148,149,150), there are many other factors that influence the degree of plastic shrinkage cracking in any particular concrete placement(151), and it is arguable that attention to these environmental and finishing factors is as justifiable for preventing plastic shrinkage cracking as adding 0.1% fibres.

More recently, there have been attempts to utilize high volumes of polypropylene fibres in shotcrete applications where steel fibre has proved very successful, as in support linings for mines, tunnels, and rock slopes, construction of thin-section elements like domes, canals, culverts and tanks, and repair of deteriorated concrete structures such as bridges(139,152). Generally, the fibre volume is 0.4-0.7%, and the wet-mix shotcrete process is preferred(132,139,152).

Carbon Fibres

Carbon fibres can be produced either by carbonizing acrylic (polyacrilonitrile or rayon) fibres at high temperature to give a high-strength high-modulus fibre, or more cheaply from the pitch extracted from petroleum or coal to give a lower modulus fibre (Table 1). Initially, efforts were made to use the high-modulus expensive variety as continuous fibre reinforcement requiring hand or automated layup of the reinforcing layers and subsequent impregnation with cement paste(153,154). The process was cumbersome, proper wetting of the fibres was difficult(17), and they were expensive.

Recently, interest has centred mainly on the cheaper pitch-based carbon fibres(62,155,156). These can be effectively dispersed throughout a mortar matrix using a special high efficiency mixer with a flexible base called an omni-mixer(41) with methylcellulose and antifoaming additives(155). The fibres are generally about 10mm long and have a very high aspect ratio of about 700 initially, but this decreases by 50% or more during mixing(155). Nevertheless, property improvements can be significant even in tension (Fig. 17), and are of course more noticeable in flexure (Fig. 58, 59) because of plastic stress distribution in the tension zone(156) (Fig. 60). Composite performance can exceed elastic-plastic (Fig. 58), flexural strength can double (Fig. 59), and toughness can reach more than 20 times the value for the matrix (Fig. 59).

Effective fibre dispersion of 10mm long fibres has also been reported using an ordinary mixer instead of the omni-mixer along with a water-reducing admixture, carbonyl methylcellulose dispersant, and a defoaming agent(157). The severity of shortening of the fibres during the mixing process affects improvements in strength and toughness, and depends not only on the process itself but on the composition of the matrix with respect to water-cement ratio, the presence or absence of sand, silica fume and polymer additives(155,158,159) (Fig. 59).

Applications -- Thin-section structural elements, some in the traditional domain of asbestos-cement and GFRC, comprise the majority of the applications, notably in Japan. They include removable load-bearing floor panels(155), and extruded products as asbestos-cement replacements(155). Cladding panels for a large dome-like monument and curtain-wall panels for a 37-storey building have been produced using carbon fibre-reinforced lightweight mortar cured by autoclaving to a specific gravity of 1.0 to 1.3(62,157). Provided the fibre can be cost competitive with other alternatives, it appears to offer cement or mortar-based composites free of the long-term deterioration problems that are associated with glass and many other fibres.

Other Manufactured Fibres

Considerable exploratory work has been done to determine what other manufactured fibre materials might be superior to asbestos, glass, steel and polypropylene in terms of mechanical properties and compatibility with the moist alkaline environment of cement paste (Table 1). Compatibility with established manufacturing processes such as spray-up, Hatschek, conventional concrete mixing, dry or wet-mix shotcreting, are also important.

Aramid Fibres -- Chemically described as aromatic polyamides, these fibres were developed primarily for use in ropes, sails, and tires. They are characterized by very high strength, comparatively high modulus, and excellent chemical stability, but their cost is high. Technically, they appear capable of significantly improving concrete performance in terms of strength, toughness and impact resistance(160,161) (Fig. 61), and, unlike carbon fibers, they are not easily damaged during mixing. Nevertheless, the longer high aspect ratio varieties, up to 10mm long with aspect ratio 800(161) require an omni-mixer or other high efficiency mixer. Silica fume is claimed to be advantageous in facilitating fibre dispersion and improving bond(162).

The durability of aramid fibres in cement is uncertain. One source reports satisfactory durability after two years at ambient temperature(160). Another reports loss of fibre strength especially at higher temperatures about 50°C(163).

Polyethylene Fibres -- Like polypropylene, polyethylene of normal molecular weight has a low modulus, bonds poorly to cement paste, and is highly resistant to alkalis(163). However, high density, high modulus modifications of the parent polymer have been developed with surface deformations to improve bond, and composite behavior approaching elastic-plastic can be achieved with 40mm fibres in conventionally mixed concrete matrices(164) (Fig. 62).

A polyethylene-based fibre pulp has been developed for thin-section applications that is compatible with the Hatschek process for asbestos-cement and with other processes used to produce thin-section cement or mortar composites(165).

Acrylic Fibres -- Chemically described as polyacrylonitrile, these fibres are of interest primarily for thin-section asbestoscement replacement by the Hatschek or similar processes where they are used in conjunction with secondary process fibres to promote full dispersion of the primary reinforcing fibres(166). Flexural strength is not significantly altered, but flexural toughness indices are substantially increased giving elastic-plastic or better composite behavior with 2-3% of 6mm long fibres in a cement matrix(166) (Fig. 63). Longer fibres up to 24mm used in concrete produced small increases in first-crack strength and modest increases in toughness with composite behavior not reaching elastic-plastic(167).

The durability of acrylic fibres in cement is questionable. Strength losses have been reported as slight at 22°C and significant at 50°C(163).

Polyester Fibres -- The term polyester encompasses a variety of material groups ranging from the textile fibres developed in the 1950's (terylene, dacron) to fibres in the thermoplastic subgrouping that are being considered for use in concrete. To date, the emphasis is on using small volume fractions of 0.1% or less as an alternate to polypropylene for controlling plastic shrinkage cracking in concrete flatwork. At this low concentration, no significant improvement in properties such as strength and toughness is expected.

The durability of polyesters in cement is questionable and controversial. Some show definite deterioration with rapid loss of fibre strength in cement(163). Hydrolysis and dissolution of the basic ingredient, polyethylene terephthalate(PET), in the presence of the alkali constituents of the cement is the problem(168), but it appears that the inclusion of a reactive pozzolan such as burnt montmorillonite may inhibit this deterioration(168). Whether this long-term deterioration need be a concern if the purpose of polyester fibres is merely to control early plastic shrinkage cracking is doubtful. The problem of fibre durability is complicated by claims that not all polyesters are subject to alkali attack(169), but the distinction between those that are and those that are not is unclear at present.

Natural Fibres

In the developing countries, some natural fibres are potentially attractive for reinforcement of cement matrices simply because of ready availability in large quantities at low cost(170). In the developed countries and other countries that have relied heavily on asbestos-cement in construction, natural fibres are viewed simply as replacements for a material that is now considered unacceptable for most applications because of the health hazard(171,172).

Natural fibres are derived from the seed and fruit, leaf, or stem of a wide range of plants and trees. Of the seed fibres, only coir or coconut fibre is of interest in cement-based composites(170). The more important of the leaf fibres is sisal, but there has been some interest in plantain. Stem or bast fibres of interest include jute, flax, ramie, sunn, kenaf, urena, elephant grass, possibly hemp which is more expensive(170), and cellulose from various species of wood(173,174). Many natural fibres are susceptible to rot as a result of bacterial or fungal action under moist conditions, loss of tensile strength and modulus under wet conditions, and excessive dimensional instability during cycles of wetting or drying. Most are prone to some degree of deterioration in the strongly alkaline environmental of cement paste(170,173,175) (Fig. 64). Their potential effectiveness depends on identifying those that are least affected by alkalis or moisture, and in some cases establishing fibre treatments and composite manufacturing processes that reduce the severity of any deterioration problem.

Cellulose Fibres -- Many cellulose fibres are only long enough to act as process fibres to facilitate the manufacture of cement composites with other types of fibre as the primary reinforcement, but some of the longer varieties have been examined for their merit as primary fibre reinforcement for strengthening and toughening(173,174). Fibre lengths are 1-3mm with diameters 12-70μ, depending on the pulping process(173). Quality in terms of strength and modulus also depends on source, microstructure and processing. Summerwood, for example, is much stronger than springwood even within a single species(174). Both regular moist curing and autoclave curing have been used for composite preparation, the latter presumably to inhibit the effects of moisture and alkalis.

Property improvements in the short term include only slight increases in flexural strength, but increases in toughness can be substantial depending on the characteristics of the cellulose fibre(173,174) (Fig. 65). However, long-term durability remains a concern which has been studied extensively(173,176,177,178). The general consensus is that natural weathering leads to an increase in interfacial bond which increases strength but promotes fibre breakage rather than pullout and therefore reduces toughness making the composite more brittle (Fig. 66).

Conditions of carbon dioxide enrichment accelerate the embrittlement process that may be due to decrease in the degree of polymerization of the fibres, more rapid carbonation of the matrix and other causes(176,177). Autoclaving appears to reduce the rate at which embrittlement develops(177).

Recycled wastepaper has the potential to be a useful source of cellulose fibres if available in sufficient quantity at a reasonable price. The effectiveness of fibres derived from watepaper in improving strength or toughness depends largely on fibre length and aspect ratio, which are less than their values for premium pulp as a consequence of the recycling process(179) (Fig. 67). Wastepaper and wood chips are a source of cellulose fibres that have been used in concrete wall panels and slabs for housing in Finland. The fibres are incorporated into ready-mixed concrete using a patented process(180).

Other Natural Fibres -- A 1980 survey of relevant literature(170) showed interest to be mainly confined to sisal and coir. Much of what has been said about cellulose fibres applies also to sisal, and the deterioration mechanisms due to alkalis and moisture are similar(181). Coir is inferior to sisal in terms of strength and stiffness, but appears to have some potential when composited with cement using pressure casting to densify the product and reduce its absorption(182). Information on other natural fibres is limited and difficult to access. The reader interested in natural fibres may obtain further details in two recent compilations on the subject(183,184).

FUTURE POSSIBILITIES

More and more fibre types that have become available through technological developments unrelated to the construction industry will likely be considered for fibre reinforcement of cement or concrete matrices. How many and how soon will depend on fibre cost, compatibility with present-day manufacturing techniques such as Hatschek, spray-up, concrete mixing, and shotcreting processes. Compatibility of the fibre with the moist alkaline environment within cement-based matrices, and proven durability of cracked composites where the fibres become exposed to air, ultraviolet light, sea water, deicing salts or other corrosive agents will be a high priority.

Hindsight suggests that compatibility of fibres with cement paste has been neglected for the sake of using fibres that were initially cheap and readily available, even though their potential for adverse chemical reaction with cement should have been obvious. Glass is an example where vast sums have been expended trying to develop protective mechanisms for a fibre that could reasonably have been rejected initially in favor of other chemically more compatible alternatives. Compatibility with conventional manufacturing techniques was also neglected by the producers of the early forms of steel and polypropylene fibres with smooth surfaces, uniform crossections, and aspect ratios too high to avoid fibre balling during mixing.

The fibres of the future must be engineered to minimize manufacturing difficulties in the freshly mixed state, maximize short-term property improvements in the hardened state, and sustain these improvements in the long term. Much remains to be done to develop test methods and performance-based specification criteria that will ensure these objectives are met.

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184. Sobral, H.S. (Editor), "Vegetable Plants and Their Fibres as Building Materials", RILEM Symposium Proceedings, Chapman and Hall, 1990, 392 pp.

Table 1 - Typical Fibre Characteristics (1,3,4,6,14,155-157,161,162)

Fibre Description	Typical Dimensions		Typical Volume-%^{a}	Specific Gravity	Modulus GPa	Poisson's Ratio
	Diam-μm	Length-mm				
Chrysotile Asbestos	0.02-30	1-40	10	2.55	164	0.3
E-Glass Single Fibre	8-10	-	-	2.53		
AR Glass Single Fibre	12.5	10-50	-	2.68	80	0.22
AR Glass Strand	110×650	10-40	2-8	2.68	70	-
Carbon-Steel	100-600	10-60	0.5-2.0	7.86	200	0.28
Stainless Steel	10-330	10-60	0.5-2.0	7.86	160	0.28
Fibrillated Polypropylene	500-4000	20-75	0.1-1.2	0.91	$<8^{b}$	0.46
Carbon, PAN-Type	8	10 -long^{c}	2-12	1.9	380	0.35
Carbon, Pitch-Based	15-20	3-20	2-4	1.6-1.9	30-200	0.35
Aramid Kevlar	12	3-13	1-2	1.44	124-133	0.32
Aramid HM-50	12	1-10	1-3	1.39	76	-
Polyethylene	900	40	2-4	0.96	5	-
Acrylic	12-100	6	2-6	1.17	14-19	-
Polyester	20-25	20-30	0.1-0.3	1.34-1.39	$<17^{b}$	-
Cellulose	15-80	2-7	10-20	1.2	10	-

Fibre Description	Tensile Strength-MPa	Elongation at Break-%	Creep Tendency	Compatibility	
				Cement Paste	Concrete Mixers
Chrysotile Asbestos	200-1800	2-3	Low	Good	Fair
E-Glass Single Fibre	3600	$\sim$2	Low	Poor	Poor
AR Glass Single Fibre	3600	$\sim$4	Low	Fair	Poor
AR Glass Strand	1450-1900	-	Low	Fair	Poor
Carbon-Steel	700-2000	3.5	Low	Good	Good
Stainless Steel	2100	3.0	Low	Good	Good
Fibrillated Polypropylene	400	8	Moderate	Good	Good
Carbon, PAN-Type	1800	$\sim$0.5	Low	Good	Good
Carbon, Pitch-Based	600-2000	1.5-2.5	Low	Good	Good
Aramid Kevlar	2900-3600	2.0-2.6	Low	Good	Good
Aramid HM-50	2900-3150	4.0-4.2	Low	Fair	Good
Polyethylene	200-300	3-80	Moderate	Fair	Good
Acrylic	270-1000	8-50	-	Good	Good
Polyester	800-1300	8-15	Moderate	Fair	Fair
Cellulose	300-700	-	-	Variable	Good
				Fair	-

a - Fibre content - percent by volume of composite
b - Rate dependent
c - Continuous

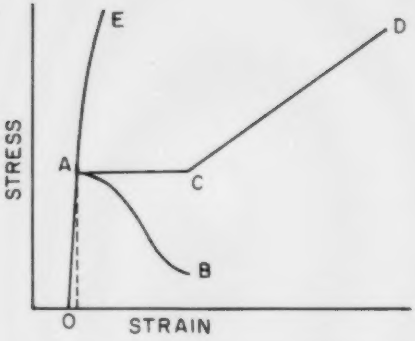


Fig. 1 Idealized Tensile Stress-Strain Curves

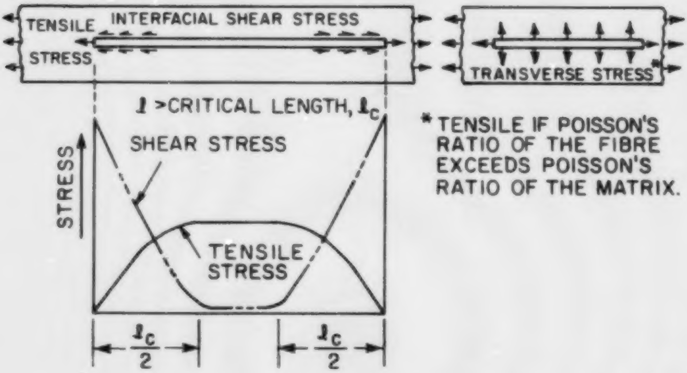

TENSILE STRESS
INTERFACIAL SHEAR STRESS
TRANSVERSE STRESS*
l >CRITICAL LENGTH, l_c
STRESS
SHEAR STRESS
TENSILE STRESS
$\frac{l_c}{2}$
$\frac{l_c}{2}$
* TENSILE IF POISSON'S RATIO OF THE FIBRE EXCEEDS POISSON'S RATIO OF THE MATRIX.

Fig. 2 Fibre-Matric Interaction in Terms of Stress Distribution and Poisson's Ratio Effect.

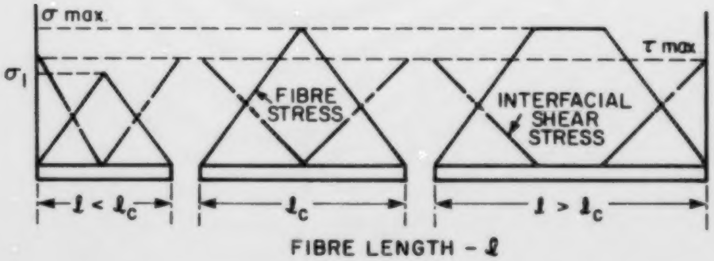
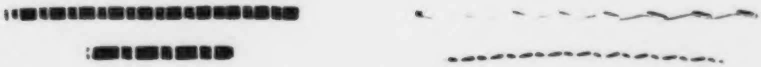

σ max
τ max
σ_1
FIBRE STRESS
INTERFACIAL SHEAR STRESS
$l < l_c$
l_c
$l > l_c$
FIBRE LENGTH - l

Fig. 3 Stress Conditions for Fibre Lengths Above and Below the Critical Length.

Surface-Deformed Wire

Enlarged-End Slit-Sheet

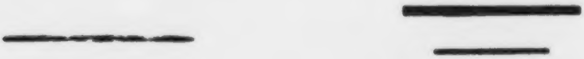


Crimped Mill-Cut

Crimped Wire

Hooked-end Wire

Twisted Mill-Cut



Carbon-Steel Melt-Extract

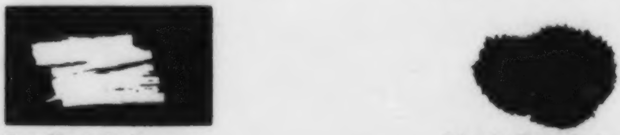
Amorphous Alloyed Melt-Extract

10mm

Fig. 4 Steel Fibres (18 mm to 60 mm)

Fibrillated Polypropylene

Alkali-Resistant Glass



Bundled Polyester

Pitch-Based Carbon

10mm

Fig. 5 Glass, Polymeric, and Carbon Fibres

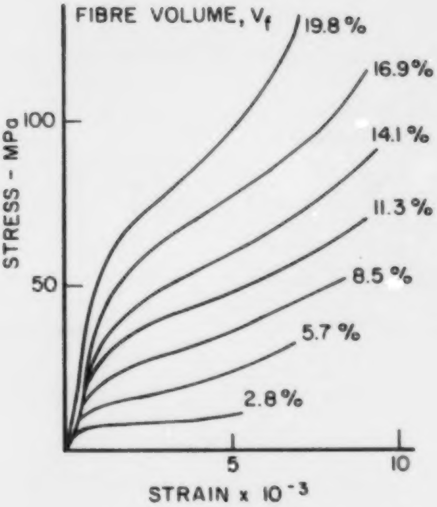


Fig. 6 Tensile Stress-Strain Curves for Glass Fibre-Reinforced Cement (GRC) with Continuous Aligned Fibres and High-Alumina Cement.

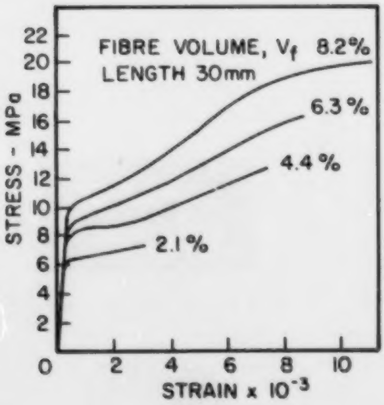


Fig. 7 Tensile Stress-Strain Curves for GRC with Different Volumes of Discontinuous Randomly Oriented Glass Fibres.

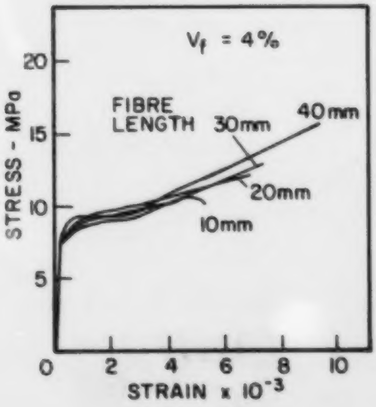


Fig. 8 Tensile Stress-Strain Curves for GRC with Different Lengths of Discontinuous Randomly Oriented Glass Fibres.

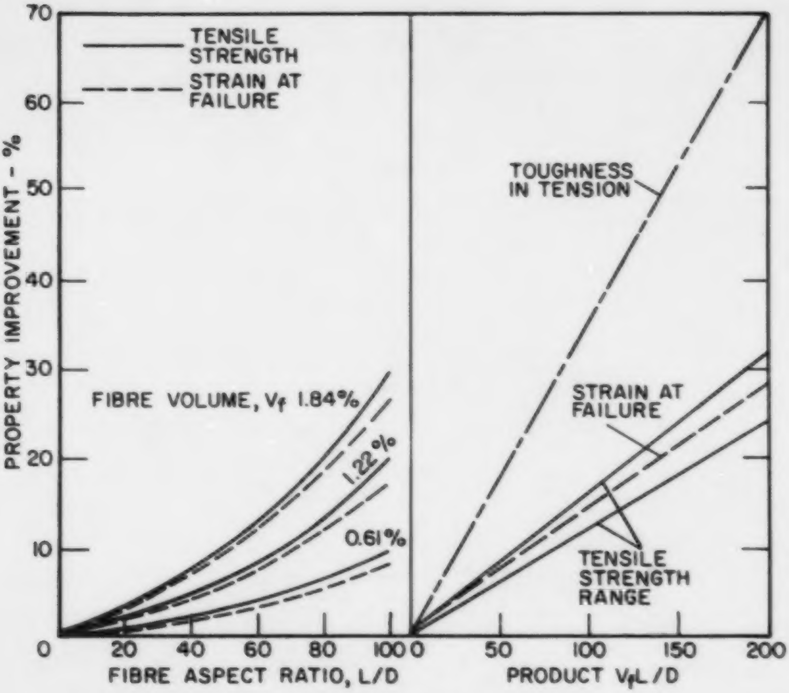


Fig. 9 Influence of Fibre Content and Aspect Ratio on Steel Fibre-Reinforced Mortar Properties in Tension.

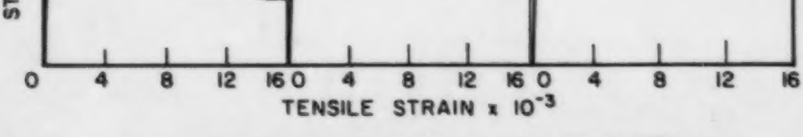


Fig. 10 Tensile Stress-Strain Curves for Steel Fibre-Reinforced Mortars with 1.73% Fibres by Volume.

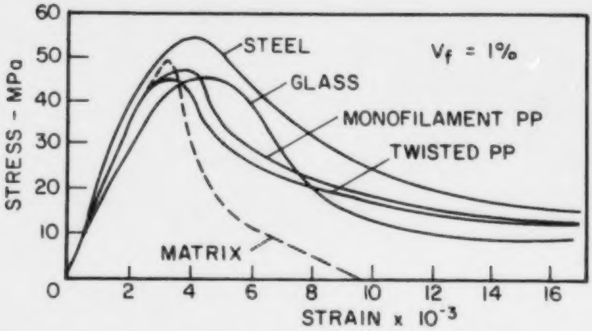
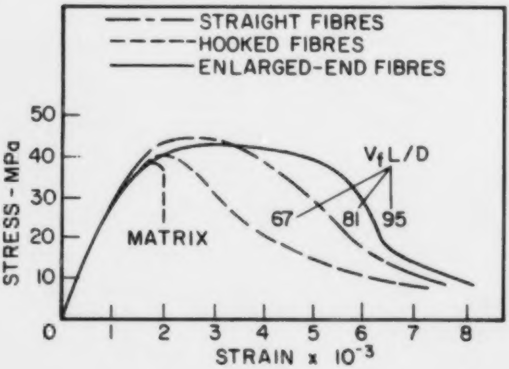


Fig. 11 Compressive Stress-Strain Curves for Fibre-Reinforced Mortars by (a) Fibre Type (above) and (b) Volume X Aspect Ratio, $V_f L/D$ (below).



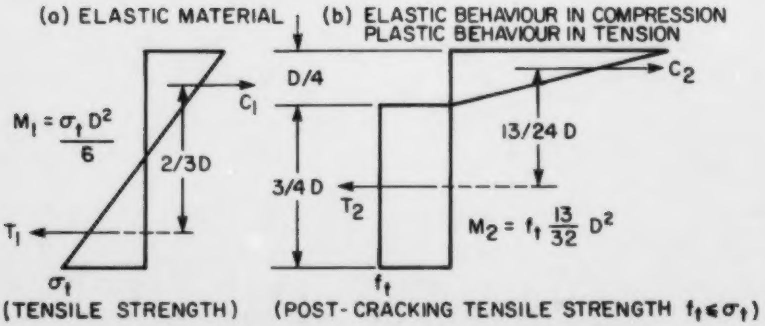

(a) ELASTIC MATERIAL
(b) ELASTIC BEHAVIOUR IN COMPRESSION PLASTIC BEHAVIOUR IN TENSION
M1 = σt D²/6
C1
2/3D
T1
σt
(TENSILE STRENGTH)
D/4
C2
13/24 D
3/4 D
T2
M2 = ft 13/32 D²
ft
(POST-CRACKING TENSILE STRENGTH ft ≤ σt)

Fig. 12 Flexural Stress Distributions for (a) Unreinforced Brittle Matrix and (b) Composites with Plastic Behavior in Tension Zone.

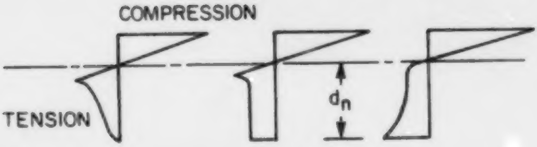


Fig. 13 Possible Stress Distributions in Flexure.

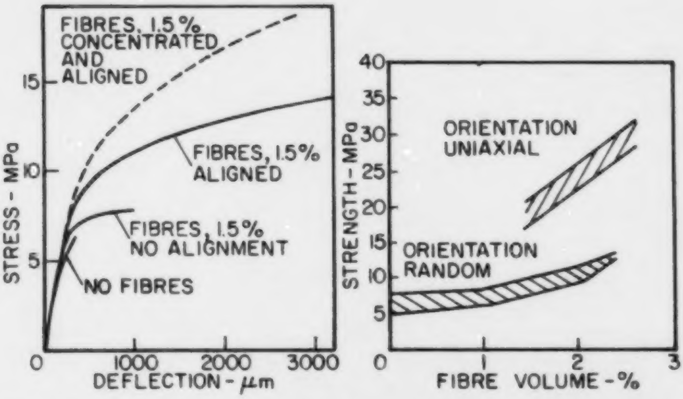


Fig. 14 Flexural Performance of Steel Fibre-Reinforced Concretes in Terms of Fibre Orientation and Distribution.

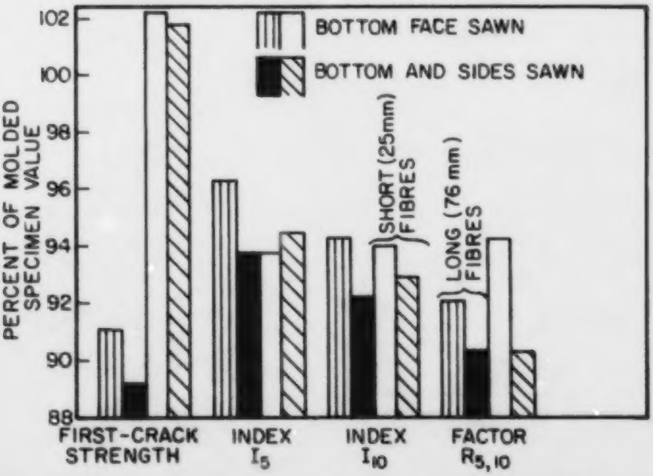


Fig. 15 Effects of Sawing the Bottom (Tension) Face and Sides of the Flexural Test Specimen on First-Crack Strength and Toughness Parameters (explained later in text).

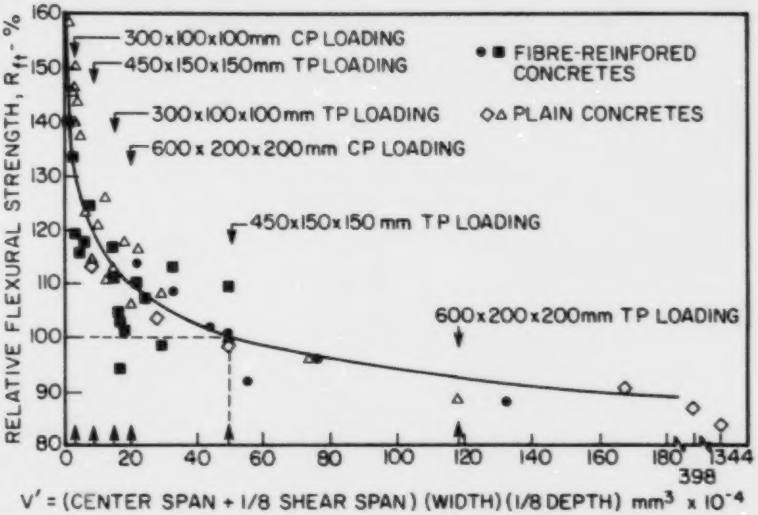


Fig. 16 Relative Flexural Strength as a Function of the Volume of the Zone of Most Probable Material Failure, V'.

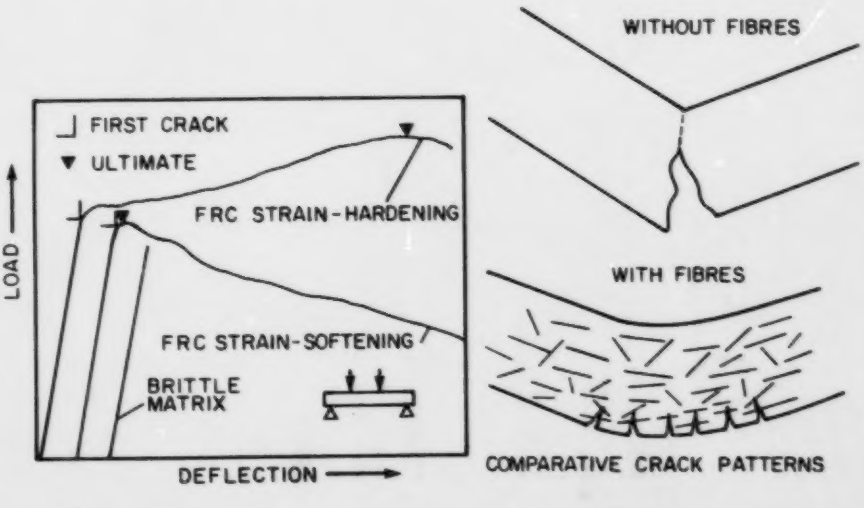

FIRST CRACK
▼ ULTIMATE
LOAD
FRC STRAIN-HARDENING
FRC STRAIN-SOFTENING
BRITTLE MATRIX
DEFLECTION
WITHOUT FIBRES
WITH FIBRES
COMPARATIVE CRACK PATTERNS

Fig. 17 Comparative Behavior of Matrix and Fibre-Reinforced Concretes in Flexure in Terms of (a) Load-Deflection Curves (left) and (b) Cracking (right).

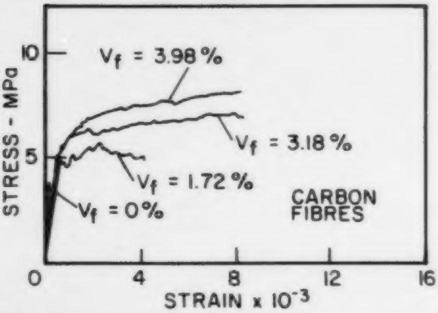
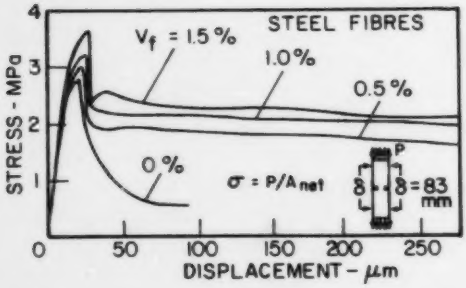


Fig. 18 Tensile Stress-Strain Curves for (a) Carbon Fibre-Reinforced Cement, Normal Testing (above) and (b) Steel Fibre-Reinforced Concrete, Closed Loop Testing (below).



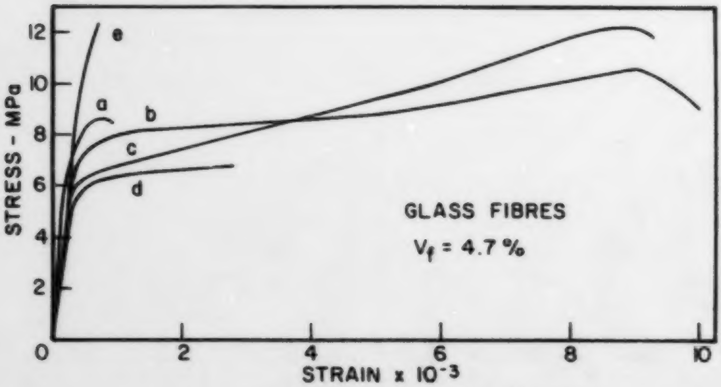


Fig. 19 Tensile Stress-Strain Curves for GRC with Various Cement Matrices After 5 Years Natural Weathering (a) Normal (b) High Alumina (c) Sulphate Resistant (d) Cement/Fly Ash (e) Polymer Modified Normal.

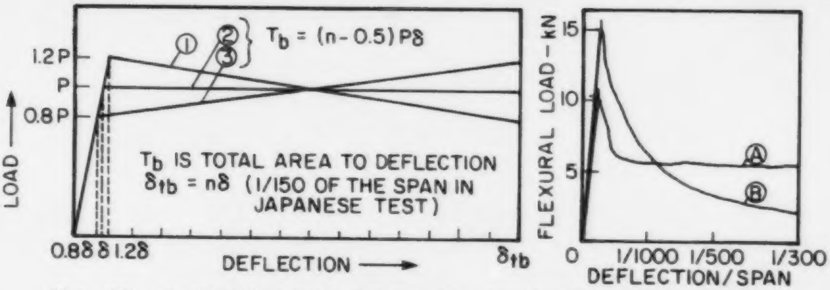
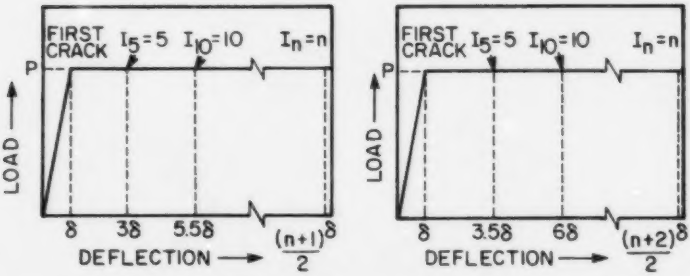
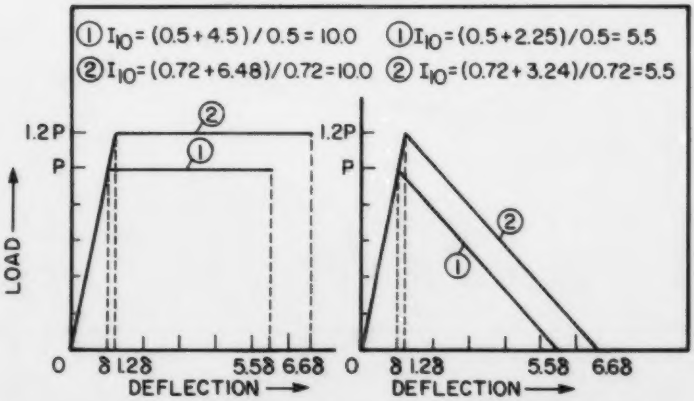


Fig. 20 Load-Deflection Curves for Equal Performance Based on Toughness (a) Schematics for Japanese Standard (left) and (b) Actual Curves for Two Different Steel Fibre Concretes (right).



FIRST CRACK
$I_5 = 5$
$I_{10} = 10$
$I_n = n$
P
LOAD
δ
3δ
5.5δ
$\frac{(n+1)}{2}\delta$
DEFLECTION
FIRST CRACK
$I_5 = 5$
$I_{10} = 10$
$I_n = n$
P
LOAD
δ
3.5δ
6δ
$\frac{(n+2)}{2}\delta$
DEFLECTION

Fig. 21 Toughness Indices Referred to Elastic-Plastic Composite Behavior (a) Base-Unity Definition System (left) and (b) Base-Zero Definition System (right).



① $I_{10} = (0.5 + 4.5)/0.5 = 10.0$
② $I_{10} = (0.72 + 6.48)/0.72 = 10.0$
① $I_{10} = (0.5 + 2.25)/0.5 = 5.5$
② $I_{10} = (0.72 + 3.24)/0.72 = 5.5$
1.2P
P
LOAD
①
②
0
δ 1.2δ
5.5δ 6.6δ
DEFLECTION
1.2P
P
①
②
0
δ 1.2δ
5.5δ 6.6δ
DEFLECTION

Fig. 22 Toughness Indices Define Average Slope of Load-Deflection Curve After First Crack Regardless of First-Crack Strength.

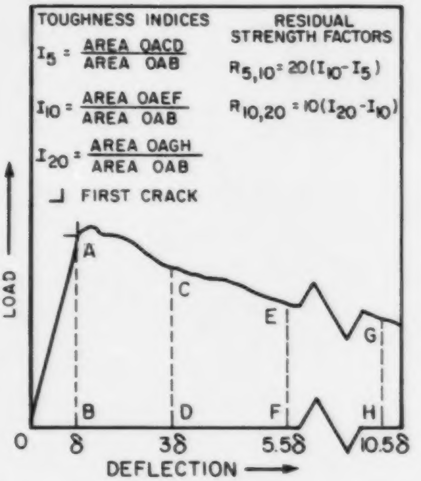

TOUGHNESS INDICES
$I_5 = \frac{\text{AREA OACD}}{\text{AREA OAB}}$
$I_{10} = \frac{\text{AREA OAEF}}{\text{AREA OAB}}$
$I_{20} = \frac{\text{AREA OAGH}}{\text{AREA OAB}}$
FIRST CRACK
RESIDUAL STRENGTH FACTORS
$R_{5,10} = 20(I_{10} - I_5)$
$R_{10,20} = 10(I_{20} - I_{10})$
LOAD
A
B
C
D
E
F
G
H
0
δ
3δ
5.5δ
10.5δ
DEFLECTION

Fig. 23 Calculation of Flexural Toughness Indices and Residual Strength Factors According to ASTM C1018.

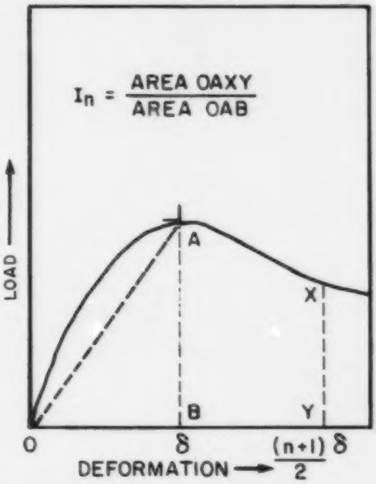

$I_n = \frac{\text{AREA OAXY}}{\text{AREA OAB}}$
LOAD
A
X
B
Y
0
δ
$\frac{(n+1)}{2}$ δ
DEFORMATION

Fig. 24 Basis for Determining Toughness Indices for Compression.

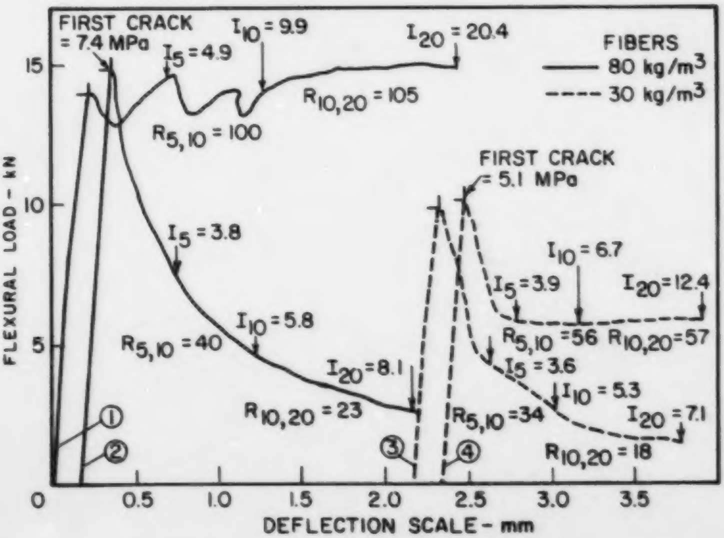


Fig. 25 Toughness Indices and Residual Strength Factors for Steel Fibre-Reinforced Concretes Using Two Different Steel Fibres and Two Different Matrix Strengths.

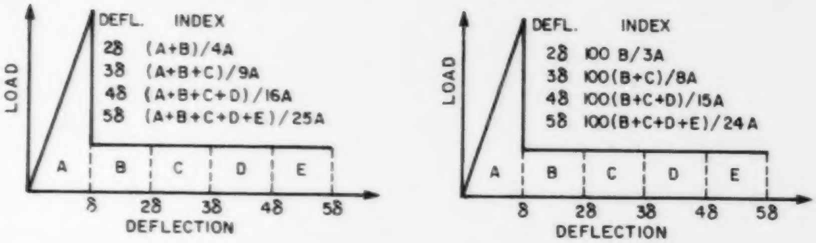

LOAD
DEFL. INDEX
2δ (A+B)/4A
3δ (A+B+C)/9A
4δ (A+B+C+D)/16A
5δ (A+B+C+D+E)/25A
A
B
C
D
E
δ
2δ
3δ
4δ
5δ
DEFLECTION
LOAD
DEFL. INDEX
2δ 100 B/3A
3δ 100(B+C)/8A
4δ 100(B+C+D)/15A
5δ 100(B+C+D+E)/24 A
A
B
C
D
E
δ
2δ
3δ
4δ
5δ
DEFLECTION

Fig. 26 Alternative Definition Systems for Toughness Indices Referenced to Linear Elastic Composite Behavior as (a) 1.0 (left) and (b) 100 (right).

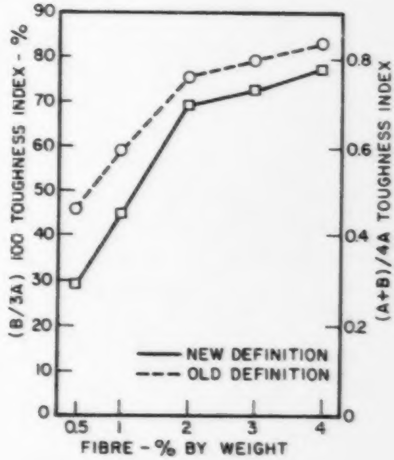


Fig. 27 Toughness Indices Versus Fibre Content for Steel Fibres of Aspect Ratio 100.

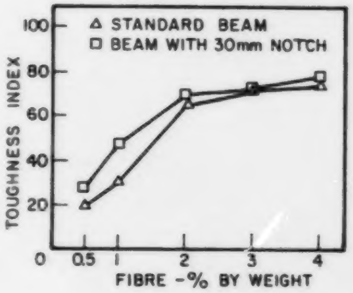


Fig. 28 Toughness Indices for Notched and Standard Beams Compared for Steel Fibre-Reinforced Concrete.

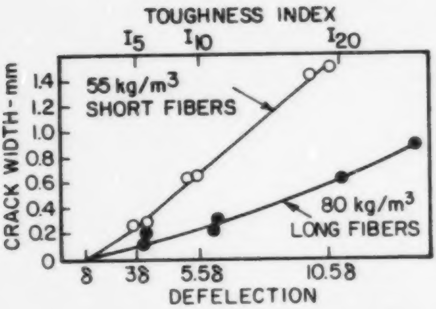


Fig. 29 Crack Development in the Flexure Toughness Test.

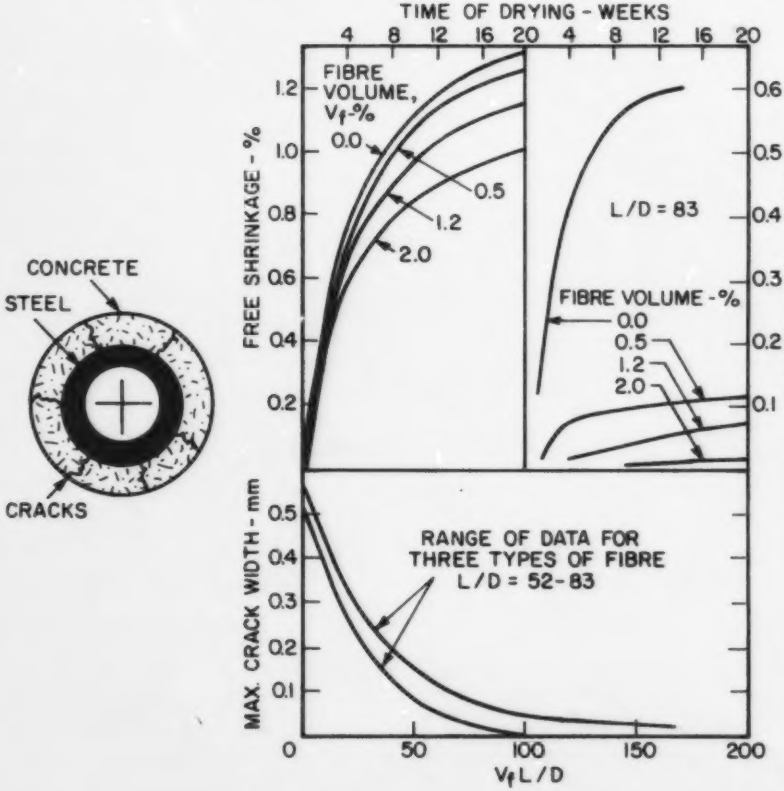


Fig. 30 Ring Test to Produce Restrained Shrinkage Cracking, (a) Schematic (left) and (b) Free Shrinkage and Crack Development for Steel Fibre Concrete (right).

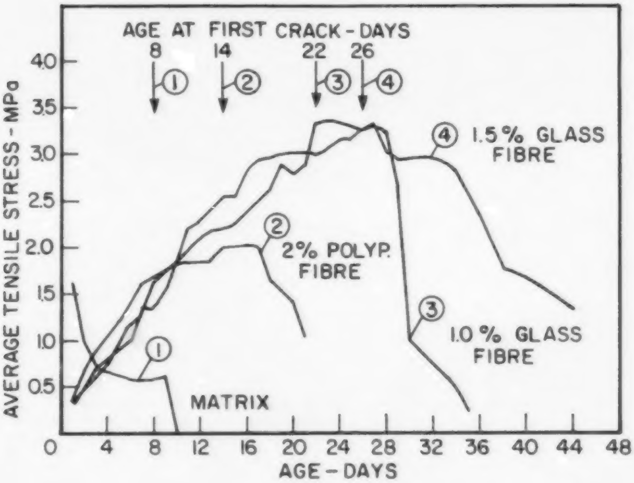


Fig. 31 Development of Tensile Stress in Fibre-Reinforced Mortar Rings under Restrained Shrinkage.

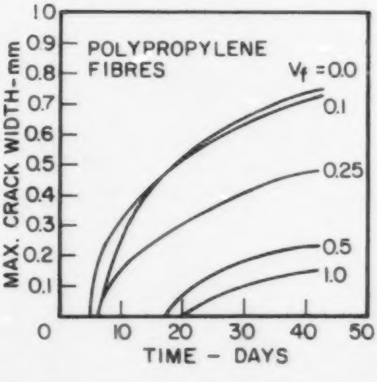


Fig. 32 Crack Development versus Fibre Volume for 19 mm Fibrillated Polypropylene Fibre-Reinforced Concrete.

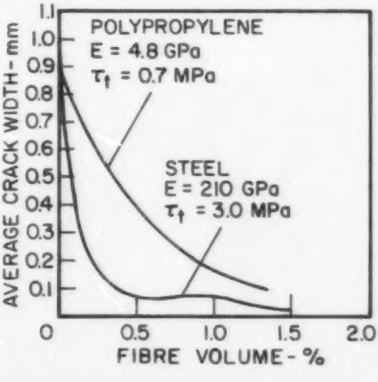


Fig. 33 Cracking Compared for 19 mm Polypropylene and 25 mm Steel Fibre-Reinforced Concrete After 6 Weeks Drying at 40% R.H.

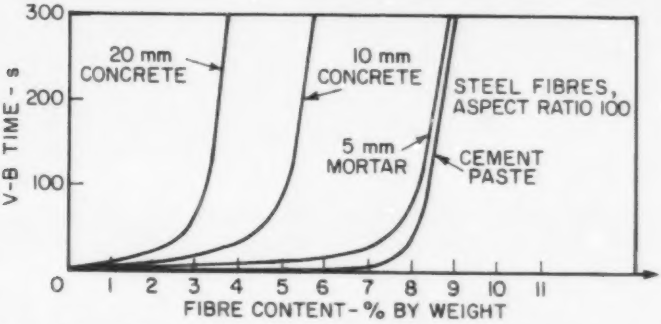


Fig. 34 Effect of Aggregate Maximum Size on Workability.

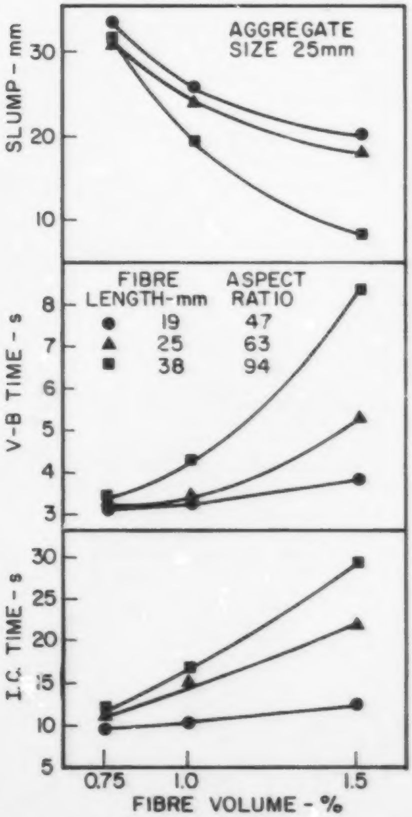


Fig. 35 Effect of Fibre Parameters on Workability of Steel Fibre-Reinforced Concrete.

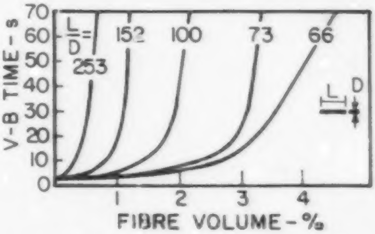


Fig. 36 Effect of Fibre Content and Aspect Ratio on V-B Time of Steel Fibre-Reinforced Concrete.

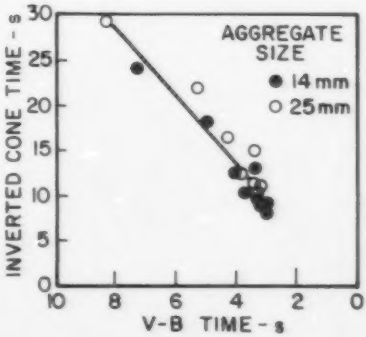


Fig. 37 Relationship Between Inverted Cone Time and V-B Time for Steel Fibre-Reinforced Concrete.

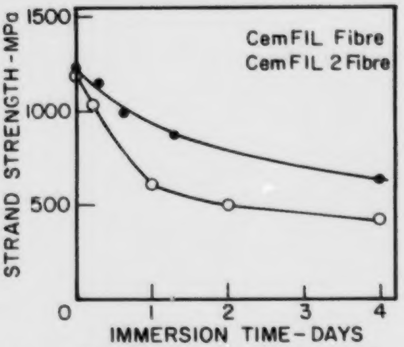


Fig. 38 Alkali-Resistant Glass Strand Strength in Cement Immersed in Water at 80°C.

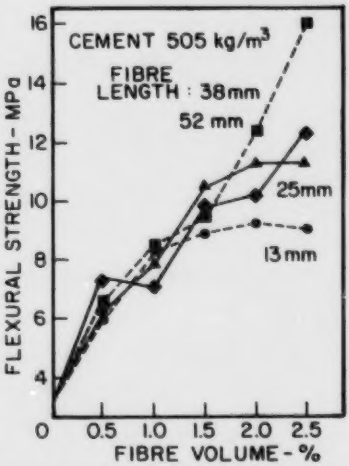


Fig. 39 Effect of Fibre Length on Flexural Strength of Glass Fibre-Reinforced Concrete.

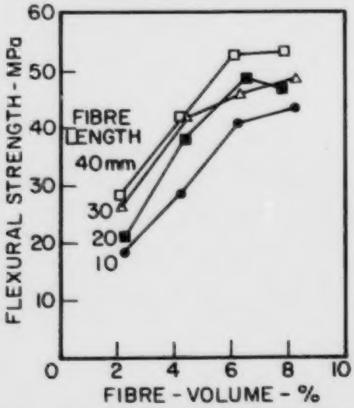


Fig. 40 Effect of Fibre Length on Flexural Strength of Glass Fibre-Reinforced Cement (GRC).

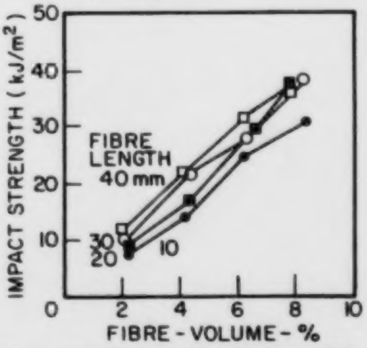


Fig. 41 Effect of Fibre Length on Impact Strength of GRC.

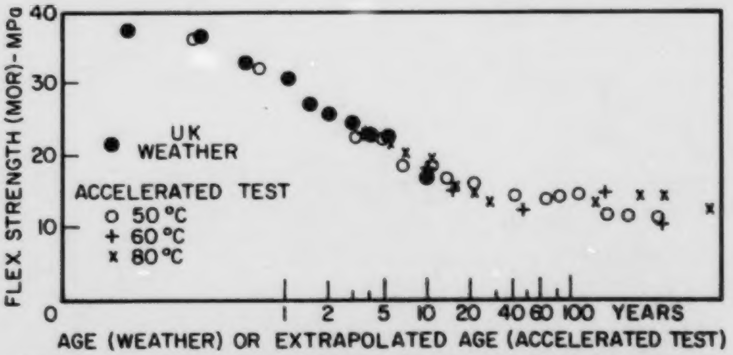


Fig. 42 Effect of Natural and Accelerated Weathering on Flexural Strength of GRC.

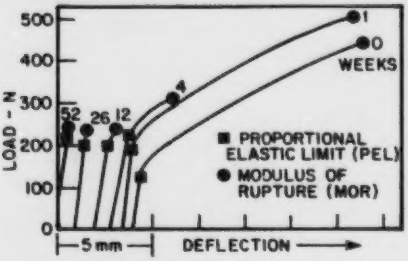


Fig. 43 Effect of Aging After 28 Days of Curing on Load-Deflection Curves of GRC.

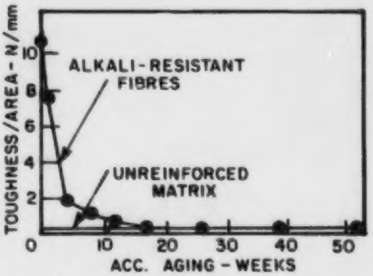


Fig. 44 Effect of Aging on Toughness of GRC.

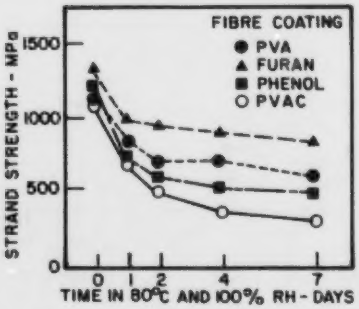


Fig. 45 Strand-in-Cement Strengths Using Fibre Coatings.

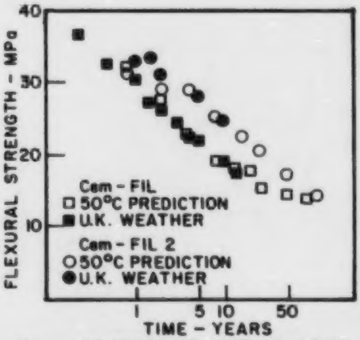


Fig. 46 Natural and Accelerated Weathering of Cem-FIL and Cem-FIL2 GRC Compared.

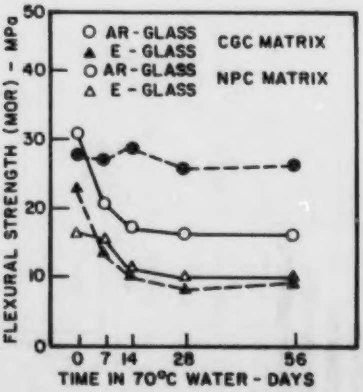


Fig. 47 Flexural Strengths of GRC Using Normal Cement (NPC) and Special Cement (CGC) Matrices.

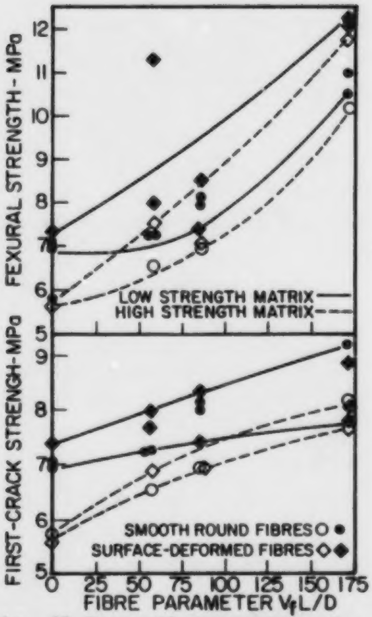


Fig. 48 Strength Parameters for Steel Fibre-Reinforced Mortar.

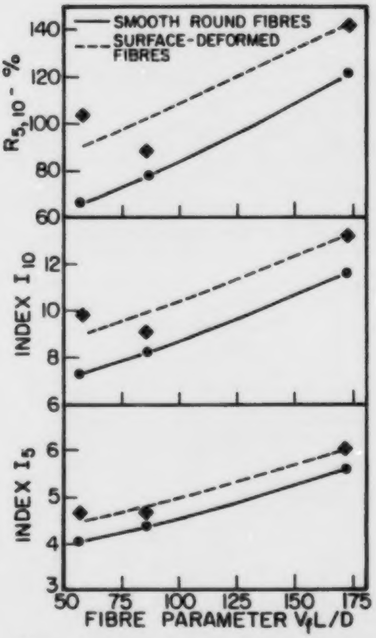


Fig. 49 Toughness Parameters for Steel Fibre-Reinforced Mortar.

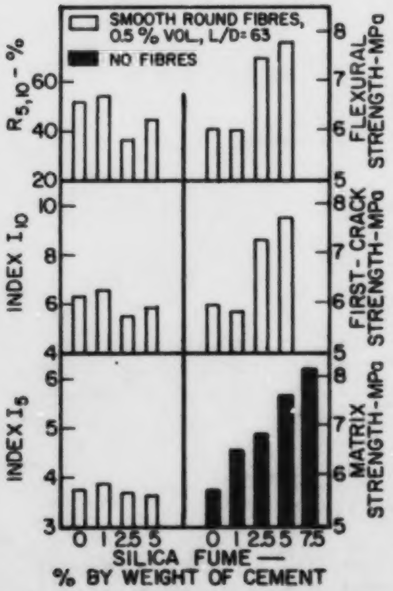


Fig. 50 Effect of Silica Fume in Steel Fibre-Reinforced Concrete.

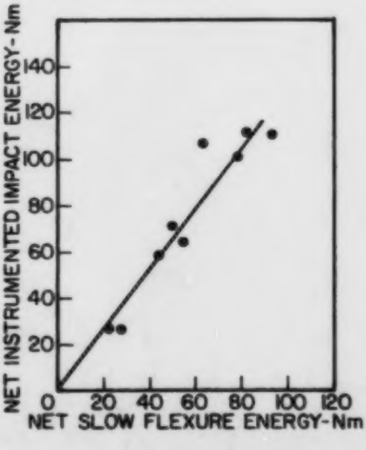


Fig. 51 Flexural Impact and Slow Flexure Loadings Compared for Steel Fibre-Reinforced Concrete.

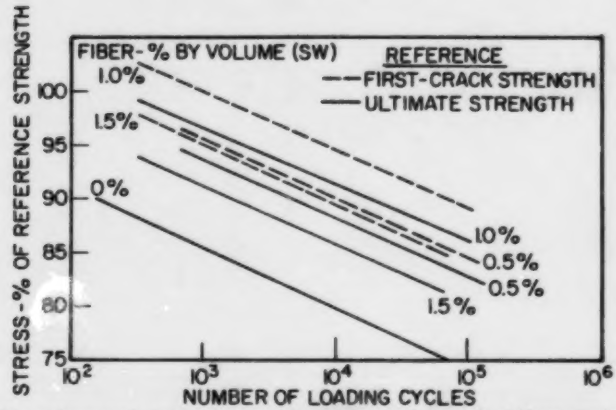
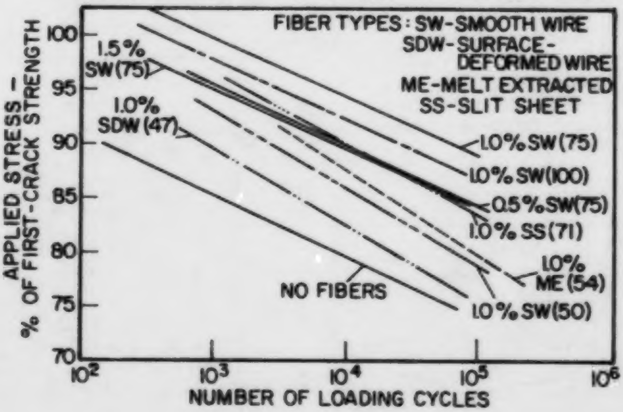


Fig. 52 Flexural Fatigue Performance of Steel Fibre-Reinforced Concrete, (a) Effect of Fibre Volume (above) and (b) Effect of Fibre Type and Aspect Radio (below).



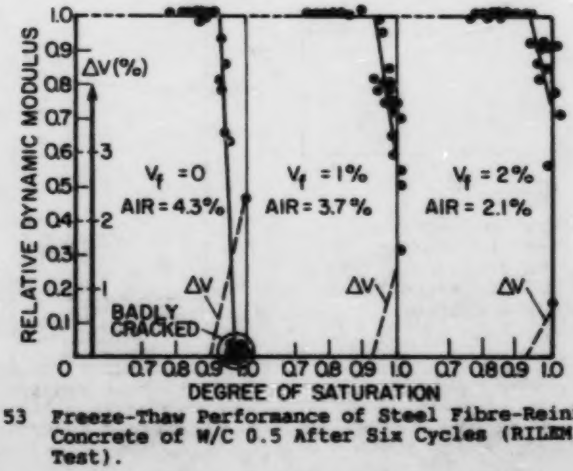


Fig. 53 Freeze-Thaw Performance of Steel Fibre-Reinforced Concrete of W/C 0.5 After Six Cycles (RILEM Standard Test).

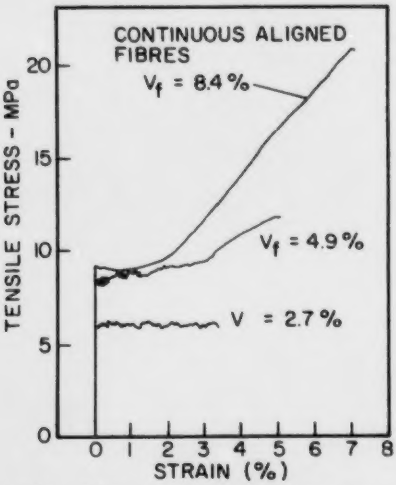


Fig. 54 Tensile Stress-Strain Curves for Cross-Ply Continuous Polypropylene Fibre-Reinforced Cement.

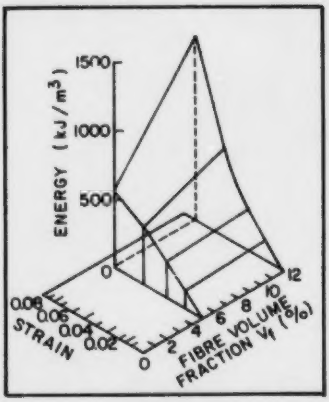


Fig. 55 Energy Absorption for Continuous Polypropylene Fibre-Reinforced Cement.

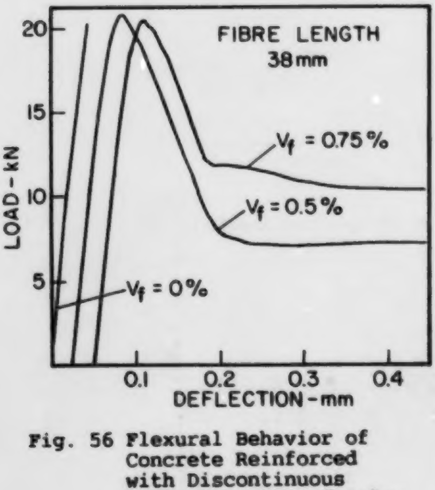


Fig. 56 Flexural Behavior of Concrete Reinforced with Discontinuous Randomly Oriented Polypropylene Fibres.

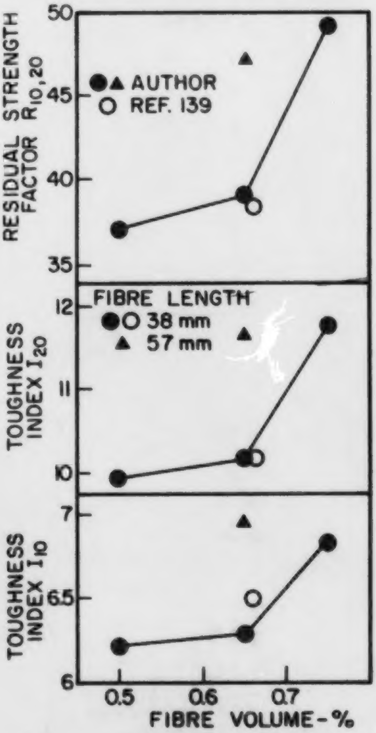


Fig. 57 Flexural Toughness Parameters for Polypropylene Fibre-Reinforced Concrete.

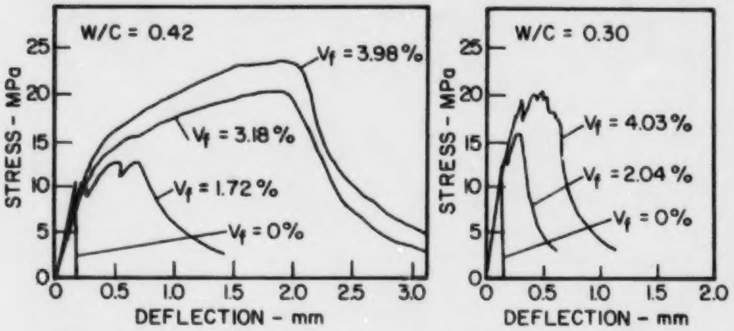


Fig. 58 Flexural Behavior of Carbon Fibre-Reinforced Cement Pastes.

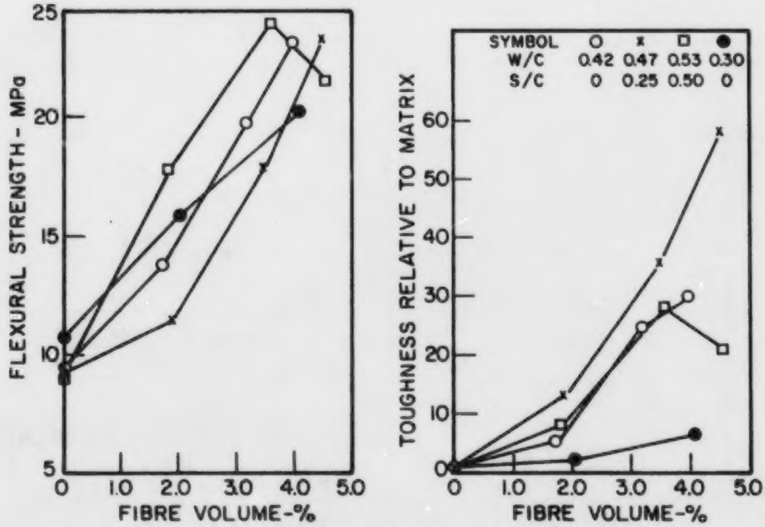


Fig. 59 Flexural Strength and Toughness of Carbon Fibre-Reinforced Cement Pastes and Mortars (10 mm Long Fibres of Aspect Ratio 690).

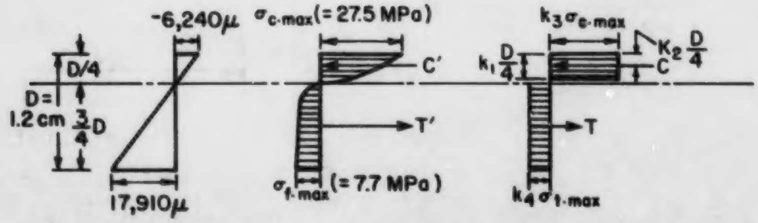

-6,240μ
D/4
D= 1.2 cm
3/4 D
17,910μ
$\sigma_{c\cdot max}$(= 27.5 MPa)
C'
T'
$\sigma_{t\cdot max}$(= 7.7 MPa)
$k_3\sigma_{c\cdot max}$
$K_2\frac{D}{4}$
$k_1\frac{D}{4}$
C
T
$k_4\sigma_{t\cdot max}$

Fig. 60 Measured Strain and Stress Distributions Compared with Idealized Distribution.

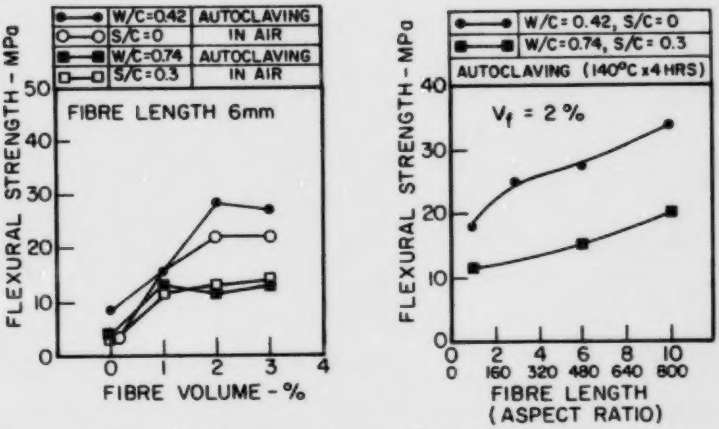


Fig. 61 Flexural Strength of Aramid Fibre-Reinforced Cement Pastes and Mortars in Terms of (a) Fibre Content (left) and (b) Aspect Ratio (right).

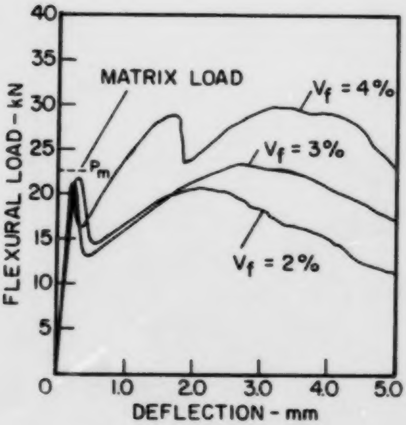


Fig. 62 Load-Deflection Curves for Polyethylene Fibre-Reinforced Concrete (40 x 0.9 mm Monofilaments).

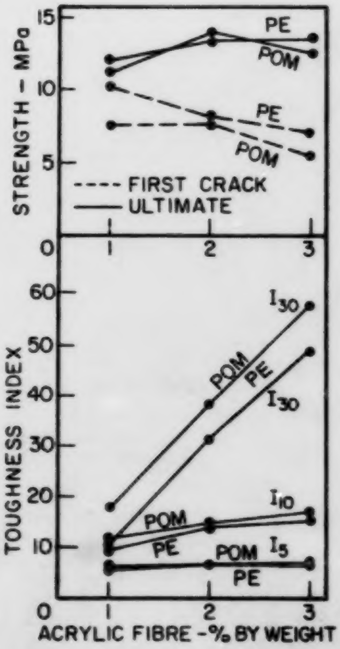


Fig. 63 Flexural Performance of Acrylic Fibre-Reinforced Cement with Cellulose and Polyoxylmethylene (POM) or Polyethylene (PE) Process Fibres.

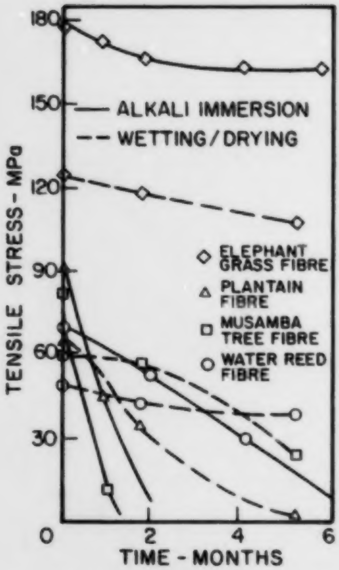


Fig. 64 Loss of Strength of Natural Fibres in Alkali or After Wetting and Drying.

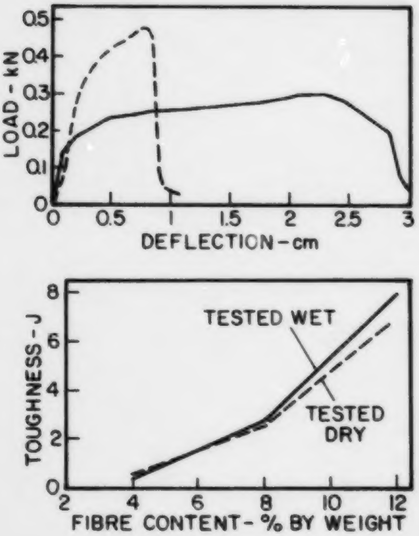


Fig. 65 Flexural Performance of Wood Pulp Cellulose Fibre-Reinforced Cement Using a Refined Slash Pine Summerwood Pulp.

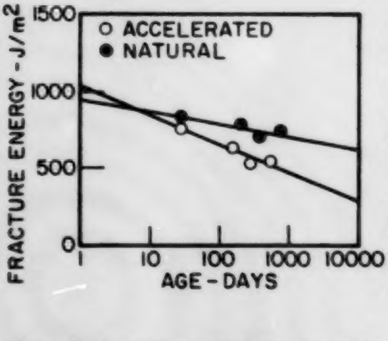


Fig. 66 Fracture Energy of Autoclaved Cellulose Fibre-Reinforced Cement under Natural and Accelerated Weathering.

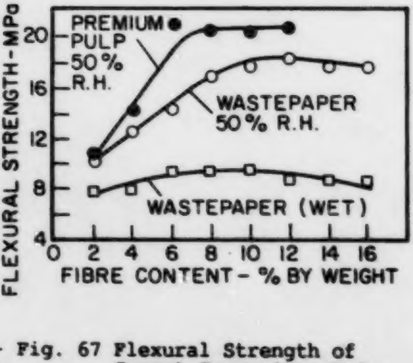


Fig. 67 Flexural Strength of Cement Composites Reinforced with Waste Paper and Premium Pulp.

New Developments in Shotcrete for Repairs and Rehabilitation

by D.R. Morgan

Synopsis: This paper presents an overview of new developments in shotcrete for repairs and rehabilitation. Consideration is mainly given to advances in shotcrete technology which have evolved in North America during the decade 1980 to 1990, although some of the advances have their origins in Europe. Prior to the 1980's most shotcrete used for repair and rehabilitation work in North America was simply comprised of plain Portland cement/sand mixtures applied by the dry-mix shotcrete process. Some polymer modified shotcrete was used for remedial works in aggressive exposure conditions and there was some limited use of the wet-mix shotcrete process, mainly in large-volume remedial projects. During the 1980's a number of new developments took place in shotcrete technology which have now found increasing application in repair and rehabilitation projects. These new developments include: the use of steel and polypropylene fibre reinforcement in lieu of conventional mesh reinforcement; the use of supplementary cementing materials such as silica fume; the use of special high early-strength cements in lieu of accelerated shotcretes. Other advances relate to improved methods for shotcrete batching, mixing and supply. Examples conclude the use of dry, premix materials in large synthetic cloth bulk bin bags and mobile volumetric batching equipment with special dispensers for materials such as steel fibre reinforcement and silica fume. There have also been significant advances in shotcrete application equipment, particularly in predampening units for use with dry premixed shotcretes and special small-line wet-mix shotcrete equipment for low-volume repairs. Finally, this paper concludes with case history examples of recent shotcrete repair and rehabilitation contracts in North America which have utilized these new developments in shotcrete technology.

Keywords: Dry mix; polymer; shotcrete; silica fume; steel and polypropylene fibres; supplementary cementing materials; wet mix.

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INTRODUCTION

In 1910 Carl Akeley, an American naturalist developed a double chambered gun for spraying a sand-cement mortar as a means of rapidly constructing moulds for museum exhibits. This process was given the proprietary name Gunite. Today this process is generally referred to as the "dry-mix shotcrete" process although the term Gunite is still used in some parts of North America. A variety of different types of guns have since been developed, including the rotary gun, and by the 1950's mixtures containing coarse aggregates were also being applied by the dry-mix shotcrete process (1). By the 1960's equipment had been developed which enabled pneumatic application of mortars and concretes; this process is now generally referred to as the "wet-mix shotcrete" process (1).

It became readily apparent to the users of both dry and wet-mix shotcrete that these systems offered advantages over conventional concrete or mortar in a variety of new construction and repair work. Shotcrete has been found to be particularly cost effective in a variety of repair situations such as the following:

- where formwork is impractical, or can be reduced or eliminated;
- where access to the work area is difficult;
- where thin layers and/or variable thickness is required;
- where normal casting techniques cannot be employed.

One of the major attributes of shotcrete is the usually excellent bond to the substrate materials produced by this process (generally superior to the bond achieved with cast-in-place repairs). This has made the shotcrete process particularly well-suited to repair of vertical and overhead surfaces. Shotcrete has thus been the selected system for repair of a wide variety of concrete structures, including deteriorated bridges, dams, sewers, reservoirs, canal linings, grain elevators, shafts, tanks, tunnels, seawalls and piers.

Prior to the 1980's most shotcrete used for repair and rehabilitation work in North America was applied by the dry-mix shotcrete process and was simply comprised of plain Portland cement/sand mixtures (2). Some polymer modified shotcrete was used for remedial works in aggressive exposure environments, particulary in the repair of bridge substructures which had deteriorated from the influence of deicing chemicals and frost attack. There was also some limited use of the wet-mix shotcrete process, mainly on large-volume remedial contracts, such as rehabilitation of dams, locks, canals and sea walls.

During the 1980's a number of new developments took place in shotcrete technology which have now found increasing application in repair and rehabilitation projects. These new developments have stemmed largely from the desire of owners, engineers and contractors to improve on the quality and durability of in-place shotcrete, increase productivity and economy of the shotcrete process and expand the range of possible shotcrete applications. These new developments include advances in shotcrete materials technology such as:

- the use of steel and polypropylene fibre reinforcement in lieu of conventional mesh reinforcement;
- the use of special high early-strength cements, in lieu of accelerated shotcretes, in projects requiring high early-strength development;
- the use of supplementary cementing materials, such as fly ash and silica fume, as additions or partial cement replacements, for the special benefits that these materials impart to shotcrete.

Other advances relate to improved methods for shotcrete batching, mixing, supply and application. Examples include:

- the use of dry-batched, premix materials, supplied in either small (typically 30 kg) paper bags or large (typically 1600 kg) synthetic cloth, bulk bin bags;

- the use of mobile volumetric batching equipment with special dispensers for materials such as steel fibre reinforcement and silica fume;

- the use of robotic shotcrete placement equipment, particularly for repair in road, rail, sewer and water tunnels;

- improvements in wet-mix pumps and dry-mix guns;

- improvements in ancillary shotcrete equipment, such as shotcrete nozzles and predampening units for use with dry, premixed shotcretes;

- special dispensing units for addition of silica fume slurry at the nozzle, with or without accelerators, in the dry-mix shotcrete process.

TERMINOLOGY

Dry-Mix Shotcrete Process

The cementitious binder (cement, with or without supplementary cementing materials) aggregates and if specified, steel fibre, are thoroughly mixed using either central mixing, transit mixer, volumetric proportioning or dry bagged premix supply. Water is added if necessary to bring the shotcrete to an "earth-dry" consistency; 3 to 6 percent moisture content. The mix is added to the delivery equipment (gun) and compressed air conveys the shotcrete down a hose a high velocity. Water is introduced under pressure through a water ring at (or near) the nozzle. The shotcrete is jetted from the nozzle at high velocity onto the receiving surface, with the force of the impacting jet compacting the shotcrete. Sometimes other materials, such as liquid accelerators, silica fume slurry, or polymer latex are added through the water ring at the nozzle (1, 2).

Wet-Mix Shotcrete Process

The wet-mix shotcrete process differs from the dry-mix process in that all the shotcrete ingredients, including mix water (but excluding accelerators) are thoroughly mixed prior to discharge into the shotcrete delivery equipment. Mixing can be accomplished by central mixing, transit mixing, volumetric proportioning or even site mixing of dry bagged premix materials. The mortar or concrete is added to the chamber of the delivery equipment and either pumped or pneumatically conveyed down the hose by compressed air. High pressure air is added at (or near) the nozzle to jet the wet-mix shotcrete at high velocity onto the receiving surface. Liquid accelerator, if required, is added through a separate line at the nozzle.

Other Shotcrete Terminology

For definitions of other terminology unique to the shotcrete process, such as: rebound, overspray, bench gunning, gun finish, flash coat, sand pocket, etc., refer to the ACI 506R-90, "Guide to Shotcrete" (1).

CONVENTIONAL SHOTCRETES

Prior to the 1980's most shotcrete mixes used in repair and rehabilitation projects were simply comprised of Portland cement, sand and water. 10 mm coarse aggregate was sometimes incorporated in the mix and on occasions materials such as polymer latex were added. In wet-mix shotcrete use was sometimes made of water-reducing and air entraining admixtures. Shotcrete accelerators were added where special project circumstances dictated their use (1).

While there are decades of experience and may examples (2) of where such shotcretes have been successfully used in repairs and rehabilitation, such shotcretes do have limitations. For example:

- Non-accelerated conventional shotcretes are limited in the thickness of build-up achievable in a single pass on vertical and overhead surfaces. Where thicker repair sections are required, the first layer of shotcrete has to be allowed to first reach initial set before successive layers can be applied. Also, the freshly placed layer has to be scarified or broomed, or if hardened, the surface has to be sandblasted or waterblasted (to remove laitance, rebound and overspray) before successive layers of shotcrete can be applied (1, 2, 3, 4). These requirements can add significantly to the cost of shotcrete construction.

- There have been examples of serious failures of shotcrete repairs where the surface of the first layer of shotcrete has not been adequately prepared prior to application of the second layer of shotcrete.

- Accelerators can be used to build up thicker layers of shotcrete in a single pass and thus avoid the need for multiple layer shotcrete construction. This technique has been extensively used in shotcrete remedial works on structures such as road, rail and water tunnels and sewers and also in remedial works in intertidal regions of marine and river structures (5, 6). There are, however, several drawbacks associated with the use of accelerators in shotcrete for remedial works, including reduced later age strength, increased drying shrinkage and hence shrinkage cracking potential and reduced durability in aggressive exposure environments (7, 8, 9), relative to non-accelerated shotcrete.

Conventional shotcrete has played a valuable role in repair and rehabilitation contracts over the 80 years since the shotcrete process was developed. It is, however, clear that there has been a need to develop shotcrete mixtures and application systems which further enhance the capabilities of the shotcrete process and provide the owner with a consistent high quality, durable, in-place final product. Some millions of dollars of funding have been expended in North America in the past decade on shotcrete research and development, both in the laboratory and the field. Much of this research and development is summarized in the references which appear at the end of this text. The sections which follow highlight some of these advances, as they pertain to shotcrete remedial work.

SHOTCRETE MATERIALS

Cementing Materials

Prior to the 1980's there was very little use in North America of cementing materials other than Portland cement in shotcrete remedial works. The exception is high alumina cement which is routinely used in refractory shotcrete applications (10). Most remedial work has been performed with ASTM Type I, II or III (Canadian Standards Association Type 10, 20 or 30) cements. ASTM Type V (CSA Type 50) sulphate resisting cements have, however, been used in remedial works subjected to sulphate exposure.

Supplementary cementing materials, such as fly ash and natural pozzolans have long been used in repair shotcretes (11). Natural pozzolans have been primarily used as pigments to provide buff, brown or red tones to shotcrete. The use of fly ash is discussed below.

Fly Ash

Fly ash has primarily been used in the past in wet-mix shotcretes. It is used for most of the same reasons it is used in concrete. For example, it is used to:

- improve the workability and pumpability of wet-mix shotcrete made with harsh aggregates;
- reduce the heat of hydration and hence potential for thermally induced cracking in thick section applications;
- improve sulphate resistance;
- control alkali-aggregate reactions;

- provide more economical mixtures.

A recent development which may find application in shotcrete remedial works is high volume fly ash wet-mix shotcrete. In such systems the fly ash constitutes between 40 to 60 percent by mass of the total cementing system and water reducers and high dosages of superplasticizers are used to produce shotcrete with a low water/cementing materials ratio (12, 13). Such shotcrete has been shown to have good volume stability (low drying shrinkage capacity), good freeze/thaw durability (when properly cured) and very low chloride permeability (as measured by the rapid chloride permeability test) relative to conventional wet-mix Portland cement shotcrete (12). These are attractive attributes for many repair and rehabilitation applications. To date such shotcrete has been mainly used for capping and sealing exposed rock slopes and bedrock (13). Further work is required to assess the performance of such high volume fly ash shotcrete in remedial works

Silica Fume

A major breakthrough in shotcrete technology was achieved with the introduction of silica fume as a supplementary cementing material. The use of silica fume in wet-mix shotcrete was pioneered in the Scandinavian countries (11) in the late 1970's and early 1980's. They found that the substitution of between 7 to 15 percent by mass of cement with silica fume imparted a number of benefits to the shotcrete process, including:

- improved adhesion and cohesion of the plastic shotcrete;
- ability to build up greater thicknesses of shotcrete with elimination of, or reduced rates of addition of accelerators;
- improved properties in the hardened shotcrete, particularly compressive and flexural strength.

In 1983 extensive laboratory and field investigations were carried out in British Columbia into the potential for use of silica fume in dry-mix shotcrete (14). Favourable feedback from these studies led to the first industrial use of the product in Canada in shotcrete repair of the 60 year old Pier B-C at Canada Place Trade and Convention Centre, Vancouver, B.C. in 1983/84. The benefits of incorporation of silica fume in dry-mix shotcrete were found to be even more pronounced than for wet-mix silica fume shotcrete (15). These benefits included:

- substantial improvement in the adhesion and cohesion of the plastic shotcrete;
- ability to build up overhead layers as much as 400 mm thick in a single pass with little or no accelerator addition;
- improved resistance to "wash-out" where freshly applied shotcrete was subjected to running water or tidal cycling;
- improved economy through substantial reductions in rebound and increased rates of productivity with single layer construction;
- improvements in the properties of the hardened shotcrete; e.g. compressive and flexural strength and freeze/thaw resistance;
- improved resistance to chemical attack and penetration of deicing salts.

Tens of thousands of tons of dry-mix silica fume shotcrete have since been used in a wide variety of shotcrete infrastructure remedial works in Canada and increasingly elsewhere in North America (14, 15).

Wet-mix silica fume shotcrete found its first application in Canada in construction of an artificial West Coast rockscape at the killer whale pool at the Vancouver Public Aquarium, in 1985. This system has since been used for a wide variety of new construction applications (14) as well as infrastructure repair; e.g. repair of berth faces in tidal zones at the Port of Saint John, New Brunswick (16).

Accelerators

As previously mentioned shotcrete accelerators have been used in remedial works to:

- increase the thickness of build-up in a single pass and thus enhance productivity;
- reduce the time to initial set and increase early (8 to 24 hour) strength development.

While accelerators can be quite effective in imparting these characteristics to shotcrete, there is a penalty to be paid. Their use nearly always results in reduced strength at later ages (5, 8, 15, 17) and the permeability and durability of the shotcrete can be adversely affected (8, 9). The extent of strength reduction depends on a variety of factors, including the particular type and addition rate of accelerator used. Later age strength reductions tend to be most pronounced with accelerator types and addition rates which promote flash setting (5, 17). Figure 1 shows the influence of a particular accelerator on the setting time of wet-mix shotcrete (5). Figure 2 shows the resulting reduction in compressive strength with increasing accelerator dosage. There is almost a 50 percent reduction in compressive strength at 28 days in the mixture with 6 percent accelerator which displayed rapid setting characteristics.

Also, it is well-known that most accelerators will significantly increase the drying shrinkage capacity of Portland cement concretes (18). Field observations have verified this effect in shotcrete. In a systematic examination of 60 shotcrete repaired bridges in Canada, the widest restrained shrinkage cracks were found in shotcrete repairs which used accelerators (7). Figure 3 shows cracks, up to 1.25 mm wide, in a bridge abutment in Alberta repaired with an accelerated dry-mix shotcrete.

Fortunately, with the advent of silica fume, it is no longer necessary to rely solely on accelerators for achieving thick layers of shotcrete build-up and single lift construction. Silica fume does, however, not substantially reduce the time to initial and final set of Portland cement shotcrete; it may still be necessary to use some accelerator where the project requires rapid set and high early (8 hour) compressive strength. Alternatively, special high early strength cements, such as described in the next section, should be used.

As a general recommendation, accelerators should not be used in shotcrete remedial works, unless considered essential to the construction process.

High Early Strength Cements

An alternative to the use of shotcrete accelerators is the use of regulated set and high early strength cements. These cements are commercially available in North America and are starting to find application in shotcrete (19, 20). A range of different cements are available, with different setting times. The most rapid setting cements are more suited to use in site-batched shotcretes; they cannot be used in central mix or transit mix shotcretes with long retention times prior to discharge. Other cements, such as the high early strength blended cement used in Reference 19 are available with either 30 minute or 90 minute setting times. This particular cement, however, can only be used in the wet-mix shotcrete process. The cement requires at least a 10 minute period of contact with water and mixing to mobilize the plasticizers and alkali activators in the formulation which give the shotcrete its high early strength development characteristics. The contact time with mix water in the dry-mix shotcrete process is too short to properly activate the cement.

Extensive testing (19, 20) has shown that these regulated set and high early strength blended cements develop higher early and later age strengths than chemically accelerated shotcretes. Also, they have shown good durability in freeze/thaw exposure environments. They thus are well suited to use in repair and rehabilitation applications where both high early strength development and long term durability are required.

Aggregates

ACI 506R-90 (1) recognizes three different shotcrete aggregate gradations. These are given in Table 1. Gradation No. 1 is essentially a concrete sand gradation and is commonly used in thin shotcrete repairs, or as a finish coat over coarser aggregate shotcrete repairs. Gradation No. 2 is a coarser gradation and typically is comprised of concrete sand and up to about 30 percent by mass of a 10 mm maximum size aggregate. Gradation No. 2 is generally recommended for use in repairs with depths greater than 50 mm. Gradation No. 3 contains up to 20 mm maximum size aggregate and is seldom used in shotcrete remedial works. It is mainly used in thick-section new shotcrete construction with predominantly downward shooting.

Other than in refractory shotcrete there has been generally little use of light-weight aggregates in remedial shotcrete. Some innovative new light-weight fine aggregates, which are reported to produce very low rebound, are, however, now being developed and may be attractive for use in remedial works.

Shotcrete aggregates should, of course, conform to all the performance requirements of national concrete standards, such as the National Standard of Canada, CAN3-A23.1-M77 or ASTM C33. In particular, aggregates proposed for use in shotcrete in remedial applications subjected to freeze/thaw cycling, or exposure to deicing salts, should be durable in these aggressive environments.

REINFORCEMENT

Conventional Reinforcement

Plain, unreinforced shotcrete, like unreinforced concrete, is a relatively brittle material, with a limited capacity to resist cracking from pronounced stresses and strains caused by influences such as:

- restrained drying shrinkage;
- daily and seasonal thermal stresses;
- deformations from load or settlement induced deformations.

Consequently in most shotcrete remedial works it is necessary to design the repair to incorporate some measure of reinforcing. Prior to the 1980's most shotcrete was reinforced with either conventional concrete reinforcing steel or welded wire mesh fabric, although in some instances even chain link mesh or chicken mesh have been used. These latter two types of reinforcement are not recommended. Expanded diamond metal lath has also been used, but in more "ferro-cement" type applications. Conventional reinforcing steel and welded wire mesh fabric remain the most common means of reinforcement in shotcrete repairs.

Steel Fibre

In the early 1970's there was a major advance in shotcrete technology with the development of steel fibre reinforced shotcrete (SFRS) [21]. While this technology had its origins in the development of pseudo-ductile shotcrete for use in lining rock slopes and support of underground openings in tunnels and mines, SFRS soon started to find applications in shotcrete remedial works (6, 22, 23). SFRS has proven to be particularly attractive for use in remedial works in aggressive chemical or marine environments, because of the enhanced resistance to deterioration from corrosion in SFRS compared to shotcrete with conventional reinforcing steel (6, 7, 16, 23).

It has been found that steel fibre reinforced concrete (24) and shotcrete (16) will not support the classic galvanic corrosion cells which can develop in conventionally reinforced concrete. As long as the matrix retains its inherent alkalinity and remains uncracked, deterioration of SFRS is not likely to occur. Corrosion of the discreet steel fibres will only occur to the depth of surface carbonation in the shotcrete; in a good quality concrete or shotcrete this need be no more than a few millimetres over a period of many years (24). If corrosion of such surface fibres is aesthetically objectionable, then a flash coat of plain, unreinforced shotcrete can be used. This technique was used with good success in the Vancouver Public Aquarium West Coast Rockscape Project (14).

Another major advantage of SFRS in remedial work is the elimination of problems of "shadowing" and voiding which are sometimes encountered in conventionally reinforced shotcrete repairs. An extreme example of this problem is shown in Figure 4. Such defects usually occur as a result of one or more of the following factors:

- too low air pressure at the nozzle;
- nozzle too far away from the receiving surface;
- shotcrete directed at an incorrect angle to the receiving surface;
- failure to properly control and deal with fresh and/or hardened overspray or rebound.

In short, SFRS has been found to be more "user friendly" and less prone to problems (such as listed above) caused by inadequate workmanship.

Polypropylene Fibre

Polypropylene fibre has started to find increasing use in shotcrete remedial works. Both discreet monofilament fibres and collated, fibrillated fibres have been used. Initially polypropylene fibre was added at the same addition rates as used in ready mix concrete; i.e. 1 kg/m^3. At this addition rate the benefits of polypropylene fibre addition are mainly limited to providing enhanced resistance to plastic shrinkage cracking and providing some "green-strength" to the freshly applied shotcrete; i.e. making the shotcrete less susceptible to sags and tears during shotcrete finishing operations.

Recently, however, extensive work has been conducted on high volume polypropylene fibre addition rates to wet-mix shotcrete (12, 13, 25, 26). This work has its origins in developing capping and sealing materials for covering certain sites. Sites treated have included:

- exposed pyritic slate cuts and bedrock in airfield and highway projects in Nova Scotia which are slaking and producing acidic leachate run off (13);
- protection of exposed mudstone vulnerable to slaking in dam construction in Alberta;
- covering and sealing a municipal incinerator waste ash in British Columbia, to minimize water entry from rain and snow melt and consequent generation of toxic leachates.

In such projects collated, fibrillated polypropylene fibre has been added to the wet-mix shotcrete at addition rates of between 4 to 7 kg/m^3. At these addition rates a pseudo-ductile material is produced, which can provide equivalent load vs. deformation characteristics to certain mesh and steel fibre reinforcement (12, 25). Such shotcrete has a much reduced cracking potential compared to unreinforced shotcrete or shotcrete with only 1 kg/m^3 polypropylene fibre reinforcement (27) and has a potential for use in a variety of different repair and rehabilitation applications. In British Columbia such shotcrete has been used for rehabilitating ravelling highway rock-cuts (see Figure 5), and repair of eroded bridge abutments.

MIX DESIGNS

Shotcrete mix design is governed by the same principles which govern concrete mix design; i.e. Abram's law applies and the prime factors controlling strength and quality are the water/cement ratio, degree of consolidation and (in wet-mix shotcrete) air content. Shotcrete repair mixes typically have cementitious contents in the range of 18 to 23 percent by mass of dry materials. The higher cementitious contents are generally used in the higher water demand, sanded mixes (ACI Gradation No. 1).

These cementitious contents correspond to values in the plastic shotcrete at the nozzle in the range of about 400 to 500 kg/m^3.

In place cementitious contents tend to be higher in dry-mix shotcretes, because of a higher mass of aggregate in the rebound materials than cementitious material. Total rebound in the wet-mix shotcrete process is generally so low (usually only about 5 to 8 percent) that the in-place cementitious content is similar to the as-batched quantity.

Silica fume is usually used as a Portland cement replacement. Replacement values typically range from 8 to 15 percent by mass of cement with the higher replacement levels being used in application such as the following:

- where the application is predominantly overhead;
- where thick sections have to be built up (on vertical or overhead surfaces) in a single pass;
- where shotcrete has to be applied to wet surfaces;
- where freshly applied shotcrete is subjected to running water or tidal cycling;
- where the surface to which the shotcrete is applied is vulnerable to "flapping"; e.g. loosely pinned diamond lath.

With dry-mix shotcrete, where the majority of the mix water is added at the water-ring at the nozzle, it is neither necessary, or desirable to incorporate water-reducing and/or superplasticizers in the mixture. By contrast, the use of water-reducing admixtures in conventional wet-mix shotcretes is considered desirable. In wet-mix silica fume shotcrete the use of both a water-reducer and superplasticizer is considered essential, in order to control the water demand of the mix; otherwise the use of silica fume will result in an excessively high water demand, which will be detrimental to the overall quality of the shotcrete and potentially lead to excessive restrained shrinkage cracking. Air entrainment is also considered essential in wet-mix shotcretes subjected to freeze/thaw or deicing salt exposure. Air entraining agents are not normally used, or considered necessary in dry-mix shotcrete (28), however, some recent work at Laval University in Quebec indicates some enhancement of the resistance of dry-mix shotcretes to freeze-thaw deterioration in the presence of deicing chemicals when air entraining admixture is added to the mix water supplied to the nozzle. Such shotcrete is being used for bridge repair in Quebec.

Steel fibre reinforcement addition rates generally vary in the range of 40 to 80 kg/m^3. The fibre addition rate used depends on the fibre type, aspect ratio (length/equivalent diameter) and the end application. Higher fibre addition rates tend to be used in structures subjected to severe stresses and strains imposed by factors such as:

- impact or explosive forces;
- heavy, repeated, dynamic cyclic loading;
- large exposed surfaces where it is important to minimize restrained drying shrinkage cracking.

Polypropylene fibre reinforcement has been used in wet-mix shotcrete remedial works. It is added at addition rates ranging between 1 and 6 kg/m^3, as previously discussed. There has been some limited use of low volume (1 kg/m^3 and less) addition of polypropylene fibre to dry-mix shotcrete.

Accelerators should only be used in remedial works where there is a specific requirement for rapid early set and/or early strength gain. A good example is the repair of transmission tower caissons in the Skeena River Estuary (6) shown in Figure 6, where the freshly applied shotcrete was subjected to strong wave and tidal action and impact from floating logs and debris. When used in remedial works, accelerators must be used with caution, so as to not excessively degrade the long-term quality of the repair.

Typical dry-mix and wet-mix shotcrete mix designs used in infrastructure remedial works in Canada are given in Table 2 (5).

BATCHING, MIXING AND SUPPLY

Dry-Mix Shotcrete

Dry-mix shotcrete for remedial works has been batched, mixed and supplied by all of the following systems:

- central or transit mixed batched, with transit mix supply;
- site-batching, using either volumetric or mass batching techniques;
- dry bagged premix materials, supplied in either small paper bags (30 kg typical) or large synthetic cloth bulk bin bags (1600 kg typical).

While quality dry-mix shotcrete has been supplied by all of the above systems, the preferred system for remedial works in Canada is dry-bagged premix (5, 14). This system has the major advantage of precision mass batching, uniformity of mixing and simplicity of use. All the ingredients are supplied premixed in the bag, including steel fibre reinforcement and, if required, dry powdered shotcrete accelerators. Figure 7 shows dry-mix, steel fibre reinforced silica fume shotcrete bulk bin bag supply for repair of the Jordan River Dam on Vancouver Island, British Columbia.

Dry bagged premix has been transported long distances by truck, train or barge, or even by helicopter, to difficult to access sites. Figure 8 shows a barge load of dry-bagged premix shotcrete on its way from a batch plant in Vancouver, B.C. to a project near Alaska. Being totally bone dry it can be used at any time after batching. The bags do, however, have to be transported and stored so as to prevent moisture from causing prehydration of the cement. Also, it is strongly advised to store the bags in protected areas during cold weather; the in place shotcrete temperature at the time of application should not be allowed to fall below 5°C; otherwise rebound tends to increase and the shotcrete can be slow in setting and developing strength.

When transit mix supply is used, the dry-mix shotcrete should not be allowed to age excessively before completion of discharge; it is generally desirable to complete discharge within 45 minutes of batching (1). In hot weather, even shorter times to completion of discharge may be required. Experience has shown that silica fume is less effective in enhancing application thickness and reducing rebound in transit mix supplied dry-mix shotcrete compared to dry bagged premix supply. It appears that the shorter the time from first contact of water with cement and silica fume to the time of application, the better the adhesive and cohesive characteristics of the plastic shotcrete.

Site-batching is still the predominant means of dry-mix shotcrete supply for remedial works in the U.S.A. Site-batched shotcrete usually has the advantage of shorter time from first contact of water with the cementing system to time of application, compared to transit mix supply. The system does, however, have the disadvantage of greater variability in mix proportioning which is sometimes encountered in site batching operations. An exception is some of the modern mobile volumetric batching equipment. When properly calibrated this equipment can produce consistently uniform quality shotcrete. Figure 9 shows a mobile batcher unit supplying dry-mix silica fume shotcrete to a bridge rehabilitation contract in Montreal, Quebec.

Wet-Mix Shotcrete

Any of the batching, mixing and supply procedures used for conventional concrete production can be used for wet-mix shotcrete. Mixtures have been batched by all of the following procedures:

- central mix batched, transit mix supply;
- transit mix batched and supply;
- volumetric site batching in mobile batcher units.

Ingredients such as silica fume and steel fibre reinforcement have been added either at the batch plant, or on site. Any accelerators used have to be added to the shotcrete at the nozzle during discharge. Silica fume has been added in all of the following ways:

- as a liquid slurry, or dry uncompacted or compacted powder, preferably at the batch plant, together with liquid superplasticizer addition;
- as a "silica fume admixture" either at the batch plant, or on site.
- as an ingredient of a blended Portland/silica fume cement (typically 8 percent by mass silica fume in the blend).

A variety of proprietary "silica fume admixtures" are available; they are supplied in both liquid slurry and dry powder form (11).

Dry powdered "silica fume admixture" addition on site is a particularly useful means of providing wet-mix silica fume shotcrete. The contractor simply orders a standard size load (say 5 m^3) of conventional plain shotcrete from a ready-mix supplier. The "Silica fume admixture" is then added directly to the transit mixer on site from either paper or bulk bin bags in a predetermined quantity. The "silica fume admixture" available from most suppliers is designed to be "neutral"; i.e. the shotcrete remains at approximately the same slump and air content after addition and mixing of the "silica fume admixture". Steel fibre reinforcement can similarly be added to the transit mixer on site, if required.

The major disadvantage of wet-mix shotcrete supply for infrastructure remedial works is that large areas have to be prepared in advance for shotcrete application and all the shotcrete has to be discharged in a limited time period (generally two hours from the time of batching at most, and often more rapidly in warm weather). Dry-mix shotcrete by contrast lends itself more readily to the smaller volumes and intermittent stop-start operations so often encountered in remedial works. The exception to this is the so-called "thin-stream" wet-mix shotcrete system. In this process small volumes of wet-mix shotcrete are continuously site batched and fed into a double chamber pressurized vessel. The discharge hose is also pressurized and the shotcrete nozzleman applies the shotcrete by simply opening a valve on the nozzle. This "thin-stream" method has also proven popular for low volume application rates of specialty shotcrete mixtures such as latex modified shotcretes.

PREPARATION FOR SHOTCRETING

As in any repair system that involves bonding of the repair material to a substrate, proper preparation of the substrate to receive the repair material is vital to the performance and longevity of the repair. It is imperative to completely remove all spalled, severely cracked, deteriorated, loose and unsound concrete from the existing structure before application of shotcrete. This can be accomplished by means such as chipping, scarifying, sandblasting, grinding and high pressure waterblasting (1). In cases where removal of large quantities of concrete is required, splitting off the face of concrete walls in structures such as locks, canals and dams has been accomplished using techniques such as:

- controlled explosive demolition;
- hydraulically activated splitting wedges;
- expanding chemical demolition grouts;
- diamond wire cutting (as used in dimension stone quarries).

When mechanical tools such as chipping hammers, scabblers or scarifiers are used it is highly desirable to follow up with wet grit sandblasting or high pressure (50 to 100 MPa) water jetting, to remove the so-called "bruised" layer of microfractured surface concrete left by the mechanical removal of concrete. Several studies have shown bond strength of the repaired shotcrete to be significantly improved when this is done.

When the cause of deterioration of existing concrete structures is corrosion of embedded reinforcing steel, it is important to remove all expanded corrosion product. This will usually require chipping around the embedded reinforcing steel, to provide a gap of at least 20 mm behind the steel, followed by sandblasting or high pressure waterblasting to remove corrosion product. Preparing the reinforced concrete structure in this way provides the benefit that the exposed reinforcing steel then becomes reinforcement in the repair shotcrete. This serves to:

- provide mechanical anchorage for the shotcrete repair;
- help restrain the potential for shrinkage induced cracking of the repair.

In certain concrete structures the extent of deterioration is such that it is not possible to chip back to sound concrete; e.g. dams, locks or berth faces with deep seated deterioration from causes such as alkali-aggregate reactivity or freeze/thaw attack. While reasonably good bond can be achieved with shotcrete in such structures, the problem lies in continued deterioration of the existing concrete behind. There is a real danger in such repairs of the sheets of surface material peeling off with time, with shotcrete still bonded to adhering old concrete behind as occurred in dry-mix shotcrete repairs to berth faces in the Port of Saint John, New Brunswick, completed in 1965, as shown in Figure 10. In such circumstances it is good design practice to not place any reliance on bond for the performance of the repair, but to adopt the design approach used by geotechnical engineers for "soil nailing"; i.e. design a reinforced shotcrete lining, with deep grouted dowels or anchors, capable of resisting applied loads from hydraulic or other forces, without placing any reliance on shotcrete bond. Such a system was used for repair of the berth faces at the Port of Saint John between 1986 and 1990.

In reinforced concrete structures where the depth of concrete removal is such that exposed reinforcing steel becomes part of the repair shotcrete, it is usually not necessary to provide additional mechanical anchorage. In structures where the reinforcing steel is not exposed, it is generally prudent to install some type of mechanical anchorage, such as grouted dowels and "Butterfly" anchorhead shear connectors, used in conjunction with reinforcement such as steel fibre or wire mesh. In this way the success of the repair is not totally dependent on shotcrete/concrete bond. The reinforcement also helps mitigate against restrained shrinkage cracking and provides the shotcrete with pseudo-ductility.

SHOTCRETE APPLICATION AND FINISHING

Application

Dry-mix and wet-mix shotcrete differ in that in the dry-mix shotcrete process compressed air conveys the damp shotcrete materials from the gun, down a hose, to the nozzle, where the remainder of the mix water is added; in the wet-mix shotcrete process the premixed wet shotcrete is either pumped or pneumatically conveyed down the hose to the nozzle, where compressed air is added to project the shotcrete onto the receiving surface. With both systems the quality of the final in-place shotcrete is highly dependent on the skills of the shotcrete nozzleman.

In the dry-mix shotcrete process the nozzleman has the ability to control:

- the amount of water added at the nozzle;
- shotcreting technique, including distance of the nozzle from the receiving surface and nozzle orientation;
- the gun operator (potman) can also be instructed, usually by hand signals, to either increase or decrease the volume of air and/or materials being conveyed down the hose.

Thus it can be seen that the nozzleman has a number of decisions to make during the shotcrete application process; proper nozzleman training and experience is essential if good quality in-place shotcrete repairs are to be assured. Good guidance with respect to dry-mix shotcrete nozzling technique is given in the ACI, "Guide to Shotcrete" (1) and the paper by Crom (4).

In the wet-mix shotcrete process the nozzleman only has to control:

- the amount of air added at the nozzle;
- shotcreting technique, including distance of the nozzle from the receiving surface and nozzle orientation;
- the pump operator can also be instructed to increase the rate of supply of wet-mix shotcrete to the nozzle (for most pumps).

Thus the wet-mix shotcrete nozzleman has a relatively simpler task, compared to the dry-mix nozzleman. The nozzleman, however, has no control over the consistency of the wet-mix shotcrete delivered; this must be correct prior to discharge into the shotcrete pump. Good guidance as to wet-mix shotcrete nozzling is given in the ACI, Guide to Shotcrete (1).

It is important during shotcrete application that the nozzle be held at the correct distance from the receiving surface. If the distance is too great, shotcrete will tend to build up on the face of any reinforcing steel, resulting in "shadowing" and voiding in behind the reinforcing and generally less than adequate consolidation of the shotcrete as shown in Figure 4. Similarly, holding the nozzle at an incorrect angle to the receiving surface can result is voids of incomplete consolidation, as well as drastically increasing rebound. With only the occasional exception (e.g. encasing large diameter reinforcing bars) the shotcrete stream should impinge at right angles to the receiving surface.

Rebound and Overspray

Another aspect of shotcrete repair which is unique to the shotcrete process is the control of rebound and overspray. Rebound is defined as shotcrete material leaner than the original mixture which ricochets off the receiving surface and falls to collect on the ground or other surfaces. Overspray is defined as shotcrete material deposited away from the intended receiving surface (usually carried away by nozzle air currents). While fresh overspray may be incorporated into the shotcrete work without any adverse effects, hardened overspray should always be removed. Hardened overspray can be removed by chipping, sandblasting or high pressure waterblasting. There have been some serious failures of major shotcrete repair installations (particularly latex modified shotcretes) attributed to a failure to properly deal with hardened overspray. For example in repair of a 300 m long x 18 m high dry dock in the USA, a 50 m^2 section of latex modified shotcrete applied by the dry-mix shotcrete process fell from the dry dock wall about two weeks after the first docking of a ship and draining of the dry dock. The entire shotcrete repair had to be removed and replaced. Extensive subsequent field and laboratory testing demonstrated that the prime cause of failure was improper shotcrete surface preparation prior application of the next layers of shotcrete.

Rebound is comprised predominantly of the coarser aggregate particles in the shotcrete mix, with some adhering mortar. Detailed studies (15) have shown rebound to not only have a lower cementing materials content than the original shotcrete, but also display a high degree of variability. Rebound should never be incorporated into the shotcrete work, either in its plastic or hardened state. Rebound is best removed continuously during the shotcrete process by the nozzleman's helper, using air wands, water jets, brooming or other suitable means. Figure 11 shows the nozzleman's helper removing overspray and rebound using an air-water lance, during "top-down" shotcrete repair of the upstream face of the Jordan River Dam, Vancouver Island, B.C. Once hardened, rebound is much more tedious and difficult to remove. Failure to adequately deal with rebound can result in localized failures in shotcrete repairs. The writer has examined a number of structures, particularly in aggressive freeze/thaw exposure environments, where localized failure was attributed to incorporation of rebound in the shotcrete repair.

Finishing

For many shotcrete repair applications, where aesthetics of the remedial works is not a concern, the shotcrete can be left in its natural gun finish. Where there is a requirement for the repair, shotcrete to conform to specific grades and alignment and finished surface texture, this can be achieved with appropriate finishing procedures. Good guidance is given in the ACI 506, "Guide to Shotcrete" (1) and paper by Crom (4). Finishing is generally more easily accomplished in wet-mix shotcrete than dry-mix shotcrete because of the higher consistency (workability) in wet-mix shotcrete. Excess material is trimmed, sliced or scraped to true line and grade using a rod or trowel. Guide strips, shooting wires and other devices are often used to facilitate this process. This leaves the shotcrete in what is called a natural rod finish. The finish suffices for many remedial applications, but if aesthetically required, the surface can be broomed, floated or trowelled to provide other surface textures.

Another approach to finishing is the application of a thin "flash" or finish coat, using a "sanded" mixture. The mixture is applied at a plaster-like consistency and then screeded, floated and if necessary, trowelled to provide the required surface texture. A wide variety of surface finishes can be achieved with shotcrete, varying from a sack-rubbed, cast-in-place concrete appearance, to exotic finishes simulating natural rockscapes, such as the Vancouver Public Aquarium Beluga Whale Pool Rockscape shown in Figure 12.

PROPERTIES OF PLASTIC SHOTCRETE

Dry-Mix Shotcrete

The plastic properties of dry-mix shotcrete are determined by the dry materials mix proportions and the amount of water added at the nozzle. The optimal amount of water addition is to a degree "self-policing". If too much water is added the shotcrete will tend to sag or slough. If too little water is added an excess amount of dust tends to be generated, rebound becomes excessively high and there is a danger of dry, incompletely compacted layers (usually referred to as "sand lenses") occurring in the in-place shotcrete. Dry-mix shotcrete adheres best and shows the least rebound and best hardened properties when shot at the "wettest stable consistency". An experienced shotcrete nozzleman is the best judge of this optimal state.

With dry-mix shotcrete without accelerators or silica fume, the thickness of build-up in a single pass attainable on overhead surfaces is generally limited to about 50 to 80 mm. On vertical surfaces much thicker sections can be shot in a single pass, particularly if the shotcrete consistency is kept stiff. As previously mentioned, with silica fume it is now possible to build-up substantially greater thicknesses on overhead surfaces with dry-mix shotcrete. The plastic property results of a recent systematic study (5) of both wet-mix and dry-mix shotcretes are given in Table 3. In thickness to sloughing tests the substitution of 11.5 percent by mass of cement with silica fume in a mix with a total cementing materials content of 420 kg/m^3 resulted in increases in the thickness to sloughing from 65 to 380 mm on an overhead surface and 200 to 460 mm on a vertical surface, as shown in Table 3.

The incorporation of silica fume in the mixture reduced the amount of rebound from 46 to 20 percent on an overhead surface and from 42 to 20 percent on a vertical surface, as shown in Table 3. Similar reductions in the amount of rebound achieved through the use of silica fume have been recorded on different repair and rehabilitation contracts.

Wet-Mix Shotcrete

The plastic properties of wet-mix shotcrete are largely determined by the quality of the mixture supplied to the pump. If the mixture is too wet, the nozzleman will have difficulty in making the shotcrete adhere and not slough. If the mixture is too stiff, pumping and shotcrete consolidation difficulties may be experienced. Wet-mix shotcretes have been shot at slumps varying from as low as 20 mm to as high as 150 mm. The preferred slump is, however, in the range of 40 to 80 mm. The lower end of this slump range is generally used for shotcrete applied to overhead surfaces, or thick vertical surfaces. Slumps greater than 80 mm are normally only used in predominantly downward shooting; e.g. repair of sloped canal linings or dam faces.

For wet-mix shotcretes exposed to freezing and thawing it is essential that the shotcrete be adequately air entrained (28). It is necessary to start with high air contents (usually in the range of about 9 to 12 percent) in the shotcrete supplied to the pump, as approximately half the air in the mixture is lost in the pumping, shooting and pneumatic consolidation process. In the study summarized in Table 2 the air content of the plain shotcrete reduced from 8.5 to 4.8 percent on shooting. The air content of the silica fume shotcrete started lower (because of the bubble coalescing influence of the superplasticizer added with the silica fume) and reduced from 6.4 to 3.9 percent on shooting. The as-shot air content can be measured either by shooting directly into an ASTM C231 air pressure meter base, or by removing shot material from a wall and reconsolidating it into the air pressure meter base. Guidance as to what constitute suitable air contents in plastic (and hardened) shotcrete, to provide freeze/thaw durable shotcrete, are given in Reference 12.

Wet-mix silica fume shotcrete can be applied to greater thicknesses before sloughing on both vertical and overhead surfaces, compared to plain wet-mix shotcrete, as shown in Table 3. The beneficial effect on thickness of build-up from incorporation of silica fume in the mixture, is however, not as pronounced as in the dry-mix shotcrete process.

Rebound on vertical surfaces is usually very low in properly designed and applied wet-mix shotcretes, irrespective of whether silica fume is used or not. In the study summarized in Table 3, rebound on vertical surfaces was only 3 to 4 percent and on overhead surfaces 13 to 15 percent; clearly much lower than in the dry-mix shotcrete process. This has significant economic implications for large shotcrete remedial projects.

PROPERTIES OF HARDENED SHOTCRETE

Shotcrete used in remedial works, in common with most concrete repair materials, tends to have a higher cementing materials content than conventional structural concretes. This has the following consequences for the repair shotcrete, compared to the concrete in the structure being repaired:

- compressive and flexural strengths tend to be higher;
- water demand in the shotcrete mixtures is usually greater; this results in higher drying shrinkage capacity in the repair shotcrete;
- good durability can usually be assured, provided proper mix design and application procedures are utilized.

Typical hardened properties of both wet and dry-mix shotcretes, both with and without silica fume are given in Table 4. The plastic properties for these shotcretes have already been described and are listed in Table 3. The actual mix designs used are given in Table 2.

It should be appreciated that the proportions given are "as-batched" proportions. The in-place shotcrete proportions for wet-mix shotcrete will not be significantly different because of the low rebound inherent in the wet-mix shotcrete process. In place cementing materials contents in the dry-mix shotcrete will tend to be higher than as-batched, because of aggregate rebound, as previously discussed. Some engineers have attempted to design leaner shotcrete mixtures, to counteract this influence. This, however, tends to be counterproductive, as lean shotcrete mixtures often display excessive rebound and the quality of the in-place shotcrete can be adversely affected. Concerns regarding high shrinkage capacity and the potential for restrained shrinkage cracking are best addressed by the incorporation of steel or polypropylene fibre in the mixture or the use of conventional reinforcement in the repair.

Compressive and Flexural Strength

Compressive strengths developed in dry-mix shotcretes repair shotcretes are typically high. One day values of 30 MPa are often achieved and 28 day strengths of around 50 MPa in plain shotcretes and 60 MPa in silica fume modified shotcretes are common. See for example the data in Table 4. Traditionally wet-mix shotcretes were considered inferior to dry-mix shotcretes with respect to strength potential. This was largely attributed to the normally higher water demand in wet-mix shotcrete for a given cementing materials content (and hence higher water cementing materials ratio). With the advent of superplasticizers and silica fume, however, this is no longer true. Very high compressive strengths at 28 day in excess of 100 MPa, can be produced if required in wet-mix shotcrete by using appropriate addition rates of superplasticizer and silica fume.

Flexural strengths are also usually quite high. Ratios of flexural to compressive strength are generally in the range of 11 to 14 percent. Flexural strengths of 6 to 8 MPa at 28 days are quire common in high strength wet and dry-mix shotcretes. Even higher flexural strengths can be achieved with high addition rates (60 to 80 kg/m^3) of certain types of steel fibre reinforcement.

Boiled Absorption and Volume of Permeable Voids

The ASTM C642 test for determination of boiled absorption and volume of permeable voids is not often specified or used to quantify cast-in-place concrete quality. It is, however, quite commonly used in shotcrete specifications and quality control testing as a means of quantifying the quality of the in-place shotcrete. Suggested indicators of shotcrete quality using these parameters, excerpted from Reference 15 are given in Figure 13. Based on these indicators, the plain wet-mix shotcrete in Table 4 would have "good" quality and all the other shotcretes "excellent" quality. This test is particularly useful in identifying less than adequately consolidated shotcrete, or shotcretes which have been damaged by excessive addition of accelerators. Recent research at Laval University has shown good correlation between boiled absorption and volume of permeable voids and deicing salt scaling resistance (9).

Durability and Parameters of the Air Void System

An appreciation of the potential freeze/thaw durability of in-place shotcrete can be obtained by extracting cores and determining the air content and parameters of the air void system. In dry-mix shotcrete it appears that the natural air content, specific surface and spacing factor produced by the dry-mix shotcrete process are such that freeze/thaw durable shotcrete can be produced (28) provided:

- high quality, freeze/thaw durable aggregates are used;
- high strength mix designs which produce low values for boiled absorption and permeable voids are used;
- good shotcrete nozzling technique is used and the shotcrete is free of defects such as "sand lenses", excessive voids of incomplete consolidation or entrapped rebound and hardened overspray.

With wet-mix shotcrete it is essential that air entraining admixtures are used such that the parameters of the air void system in the in-place shotcrete are satisfactory. Guidance as to what constitute acceptable parameters, for different qualities of shotcrete, are given in Reference 28. Failure to adequately air-entrain wet-mix shotcrete can result in severe failure of shotcrete exposed to freezing and thawing in a saturated condition. Extensive freeze/thaw durability testing to ASTM C666, Procedure A, both in the laboratory and several shotcrete remedial projects, have demonstrated that the production of freeze/thaw durable shotcrete is readily achievable.

Table 4 provides air content, specific surface and spacing factors for the wet and dry-mix shotcretes reported in this study. The plain wet-mix shotcrete and silica fume modified dry-mix shotcrete had spacing factors 0.25 and 0.26 mm respectively. These mixes should be freeze/thaw durable. The spacing factor of the wet silica fume mix at 0.36 mm is higher than desirable (attributed to too low an air content before shooting, as previously discussed) and this mix may not be sufficiently durable if exposed to freezing and thawing in a saturated condition. The freeze/thaw durability of plain dry-mix shotcrete, with a spacing factor of 0.35 mm is also likely to be questionable.

EXAMPLES OF SHOTCRETE REPAIR OF INFRASTRUCTURE IN NORTH AMERICA

Shotcrete has been used for decades for repair and rehabilitation of structures in North America. This section summarizes some recent reported work which has utilized one or more of the new shotcrete developments described in this paper.

Pier B-C, Canada Place, Vancouver, B.C.

The Pier B-C structure in Vancouver Harbour now serves as the substructure supporting the Pan Pacific Hotel and Canada Place Trade and Convention Centre. The original pier was constructed between 1923 and 1927 and served as the Pacific terminus for the Canadian Pacific Railway. Between 1983 and 1985 some CAN. $2 million was spent on pier remedial works. The pier consists of a central berm projecting 330 m from the shore, surrounded by a concrete deck supported on about 6000 precast reinforced concrete piles driven into the harbour.

Investigation of the structure in 1979 showed that while most of the precast piles were in still excellent condition, many of the cast-in-place reinforced concrete members had undergone extensive corrosion induced deterioration. Additional damage was done to the pier during construction of the new facilities because of differential settlements and spreading of the pier caused by installation of additional piling and caissons to support heavily loaded parts of the new structure (e.g. masts supporting the tent structure).

Extensive use was made of dry-mix shotcrete for repair of beams, piles, pilecaps, seawalls and slab soffits. Initially the work was carried out with plain transit mix supplied and some plain, bagged, dry-mix shotcrete. The use of shotcrete accelerators was not permitted and productivity difficulties arose because of limited available working time for shotcrete remedial works in intertidal regions. The known good wash-out resistance of concrete incorporating silica fume provided the impetus to experiment with the inclusion of silica fume in shotcrete for intertidal repair. Substantial improvements in wash-out resistance and other improvements in the plastic and hardened shotcrete, established in extensive laboratory and field studies (15) resulted in the Pier B-C project being completed using dry-bagged silica fume shotcrete. Performance of the shotcrete in the 7 years since installed has been excellent. The nearly completed structure is shown in Figure 14.

Port of Saint John, New Brunswick

Concrete berth faces (wharves) in the Port of Saint John are subjected to some of the most severe environmental exposure of any concrete structures in the world. With a tidal range of 8.5 metres and winter temperatures which fall as low as --30°C, the intertidal regions of the berth faces are subjected to over 200 freeze/thaw cycles most years. To make matters worse the structures were constructed long before the development of air entrainment and much of the original aggregate used in the concrete in these berth faces has been shown to be alkali reactive. The net result has been severe deterioration and loss of concrete from the berth faces from freeze/thaw attack and alkali aggregate reactivity. In some of the older (1913) structures, daylight could ben seen through 6 m of concrete, as shown in Figure 15.

Details of the remedial works undertaken on this project are given in Reference 16. Extensive preconstruction testing was undertaken to satisfy the Port Authority that the selected repair material would be durable in this aggressive environment. A variety of repair alternatives were investigated, including cast-in-place concrete, preplaced aggregate concrete and shotcrete. The shotcrete alternative proved most economic. The shotcrete was central mix batched, and transit mix supplied, with the steel fibre reinforcement being added to the transit mixer at the job site. The shotcrete was applied from a 10 m long barge, on the falling tide; the barge dropped at a rate of about 1.2 m per hour, providing an effective moving scaffold. Figure 16 shows the shotcrete being applied.

No accelerator was used, other than in dental shotcrete used to fill deep holes in the berth faces prior to application of the structural facing shotcrete. No reliance for performance of the remedial design was placed on the quality of bond of the repair shotcrete to the existing concrete, in view of the deteriorated condition of the existing concrete. Instead the existing concrete was treated almost as a rubble slope, and the structural shotcrete was anchored with 1 m long grouted dowels placed vertically and horizontally at about 1.4 m centres. Shear reinforcement was used to connect anchor heads.

Some restrained drying shrinkage cracking has occurred (as expected), but the cracks are tight, generally less than 0.1 mm wide at the surface. These crack widths are in the range considered by Hoff (24) to be sufficiently tight to provide acceptable resistance to corrosion of the steel fibres in a marine environment. Shotcrete applied in 1986 has now undergone over 1000 freeze/thaw cycles and there is no indication of freeze/thaw distress. The best testament to the selected shotcrete repair system is that the Port Authority have continued to use the same remedial system for additional berth face repair every construction season, as maintenance funds have permitted.

Jordan River Dam, Vancouver Island

The Jordan River Dam serves as water supply for peak-load hydro electricity generation for the City of Victoria, B.C. The dam is an old Ambersen buttress dam, built in 1912. The dam was constructed long before the development of air entrainment for concrete and by 1970 the upstream face of the dam and other parts of the structure vulnerable to critical saturation had undergone significant freeze/thaw deterioration. In 1971 the entire 45 degree slopes upstream face of the dam, as well as certain parts of the soffit of the buttress slab and the buttresses were repaired with a "gunite" mix; i.e. a conventional plain, sanded (ACI Gradation No. 1) dry-mix shotcrete.

In 1987 detailed investigation of the structure revealed that while the majority of the upstream face of the dam was still in reasonable condition, some deterioration was evident. This deterioration was attributed to a combination of:

- acid attack from moss-growth above the working range of the reservoir;
- freeze-thaw attack in the working range of the reservoir where the shotcrete was vulnerable to critical saturation; the freeze-thaw attack tended to be concentrated in porous areas of shotcrete, where rebound had been incorporated into the work;
- ice jacking, between successive layers of shotcrete; attributed to a failure to adequately remove hardened overspray and rebound prior to application of the next layer of shotcrete.

Remedial works were undertaken in 1989 using dry-bagged, steel fibre reinforced, silica fume shotcrete applied by the dry-mix process as shown in Figure 7. Areas of deteriorated shotcrete were removed using chipping hammers and high pressure waterblasting. Particular care was taken to remove rebound and any accumulations of overspray from the dam face during shotcreting operations as shown in Figure 11. This was accomplished using air/water lances and pressurized water hoses. Construction was completed from the top down to facilitate removal of rebound and overspray.

Some interesting failures in the 1971 shotcrete were noted on exposed portions of some of the downstream buttresses. A thin layer of dry-mix shotcrete (about 15 to 20 mm thick) had been applied as a "flash coat" to seal parts of these structures. No anchorage or reinforcement had been used in the repair shotcrete. When sounded with a hammer some of this shotcrete sounded "drummy". In places it could be barred down with a crow-bar. Bond of the shotcrete to the original concrete was generally excellent; delamination was observed to have occurred behind the shotcrete/concrete bond interface, in the old concrete. This demonstrates the importance of using anchorage systems in the remedial design.

Transmission Tower Caissons, Skeena River Estuary, B.C.

A 69 kv transmission line crosses the Skeena River estuary near Prince Rupert in Northern B.C. at a point where the estuary is 2.2 km wide. Three suspension crossing towers supported by concrete pier foundations carry the line. These piers were constructed in 1929. Twelve millimetre thick steel sheet piling used as formwork for each of the two 9 m high piers was left in place to help protect the concrete from scour by river ice. The combination of nearly 60 years of impact from river ice as well as corrosion had severely deteriorated the sheet piling. There was also extensive scouring of the concrete behind the sheet piling (6). Over 600 mm thickness of concrete had been scoured out of the upstream noses of the piers, threatening to undermine the 3 m high tie beams which span across the front of the piers. It became a matter of some urgency to repair these piers. Figure 17 shows one of these deteriorated piers.

The selected repair material had to be capable of resisting impact and scour from river ice and floating debris and be capable of placement quickly during low water tidal periods. Various repair alternatives were evaluated, but the only two methods considered practical were:

- anchor steel plates to the face of the piers and fill the void behind with cement grout;
- apply a layer of steel fibre reinforced, dry-mix, silica fume shotcrete to the surface of the piers.

Cost analyses indicated that the shotcrete repair solution was about 30 percent less expensive than the steel plate/grout alternative (6). Studies by the U.S. Corps. of Engineers have shown that steel fibre reinforcement substantially enhances the resistance of shotcrete to impact of large debris; abrasion resistance to low velocity flow has been shown to be significantly improved through the use of silica fume to produce a high strength shotcrete. The average compressive strength of shotcrete used on the project was 58 MPa at 28 days with one test panel reaching a strength of 76 MPa at 28 days.

Details of the remedial works are given in Reference 6. Dry-bagged steel fibre reinforced silica fume shotcrete was supplied to the project by barge. The contractor was able to apply the shotcrete to vertical faces at thicknesses of up to 600 mm in a single pass. Figure 6 shows this work in progress. This was considered a great cost saving advantage, as there were only short periods of low water occurring every couple of weeks. Accelerator was used in this project in order to provide rapid set, so that the freshly applied shotcrete would not be damaged by floating debris. Washout resistance of the shotcrete was excellent. No shotcrete was lost, in spite of shotcrete on the lower portions of the piers being only a few minutes old when subjected to strong currents and wave action with the rising tide. This project has verified the viability of steel fibre reinforced, silica fume shotcrete for challenging marine repair applications in intertidal zones.

Bridge Repair with Shotcrete in Alberta

In the late 1970's and early 1980's, Alberta Transportation used latex modified shotcrete for repair of bridge substructural elements (7, 23). While the latex-modified shotcrete generally produced high flexural strengths and low permeability, there were a number of negative factors, including:

- the latex was costly;
- only a few specialty contractors were capable of doing the work;
- the latex modified shotcrete was very sticky and tended to be difficult to finish;
- problems of plastic shrinkage cracking were frequently encountered.

In 1984, Alberta Transportation let contracts to repair seven bridges with steel fibre reinforced shotcrete. Reasons given for switching from latex shotcrete to steel fibre reinforced shotcrete included (23):

- a more economical repair method;
- more qualified contractors able to bid the work'
- elimination of most reinforcing mesh;
- less surface cracking.

A variety of different types of repairs were undertaken. Three bridges constructed between 1962 and 1967 required repair of load and deicing salt deteriorated precast channel shaped girders. Steel fibre reinforced shotcrete was used to repair corrosion induced spalls on the channel legs and help tie the channel sections together. Much of the work required overhead shooting.

The other four bridges required repair of concrete abutments and piers which had suffered deterioration from freeze/thaw and deicing salt attack. These bridges were quite old, having been constructed between 1922 and 1958. Most deterioration was caused by open deck joints permitting water and deicing salts to leak through and saturate piers and abutments. Remedial works involved replacement or repair of leaking deck joints followed by repair of the substructural elements. Deteriorated concrete was removed with chipping hammers and replaced with dry-bagged, steel fibre reinforced shotcrete applied by the dry-mix shotcrete process. Where aesthetics was of concern, a thin flash coat of shotcrete without fibre was applied, to prevent the appearance of rust stains in the finished surface. Surfaces were generally finished to rounded profiles to facilitate water run-off and prevent ponding. An acrylic sealer was applied to the finished work; this served both as a curing compound to minimize shrinkage cracking potential, as well as enhancing resistance to salt penetration.

Carter (23) concluded that overall the steel fibre shotcrete provides a successful, cost-effective method of repairing concrete bridge structures where moderate volumes of repair material are needed. In the late 1980's, Alberta Transportation started specifying and using silica fume in steel fibre reinforced shotcrete repairs to further enhance productivity and the quality of repairs. Figure 18 shows repairs to bridge piers at a bridge over the North Saskatchewan River at Drayton Valley completed in 1986 using such shotcrete. When examined in 1990, the repairs were observed to be essentially crack-free (7).

Miscellaneous Shotcrete Repairs

In addition to the case-history projects listed, shotcrete has been used to repair a wide range of reinforced concrete structures throughout North America. Steel fibre reinforced, silica fume shotcrete has been used to repair:

- electrostatic precipitators and other structures in pulp mills in British Columbia;
- bulk shipping facilities in Vancouver harbour e.g. potash and sulpher load-out dumper pits and conveyors;
- a variety of different dam and reservoir structures;
- numerous wharfs, piers and other marine facilities on both the Atlantic and Pacific Coasts;
- elevated expressways and traffic interchanges;
- seismic upgrading of masonry and reinforced concrete structures.

CLOSURE

Conventional shotcrete has been used for decades for repair and rehabilitation contracts. Most such repairs have performed well, but unfortunately there have been cases of failure, or less than adequate performance (7, 29, 30). Most such failures can be traced to poor construction practice, although in some cases poor design, or the use of inferior materials have contributed to the problem. The advances in shotcrete materials, batching, mixing, supply, equipment and application technology reviewed in this paper enhance the prospects for the production of more economic and durable shotcrete repairs. Continuing research will lead to the development of even more improvements in the shotcrete process and extend the range of use of shotcrete in repair and rehabilitation applications.

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Table 1: ACI 506R-90, Table 2.1, Gradation Limits for Combined Aggregates for Shotcrete

Sieve Size, U.S. Standard Square Mesh	Percent by Weight Passing Individual Sieves		
	Gradation No. 1	Gradation No. 2	Gradation No. 3
3/4 in. (19 mm)	--	--	100
1/2 in. (12 mm)	--	100	80 - 95
3/8 in. (10 mm)	100	90 - 100	70 - 90
No. 4 (4.75 mm)	95 - 100	70 - 85	50 - 70
No. 8 (2.4 mm)	80 - 100	50 - 70	35 - 55
No. 16 (1.2 mm)	50 - 85	35 - 55	20 - 40
No. 30 (600 μm)	25 - 60	20 - 35	10 - 30
No. 50 (300 μm)	10 - 30	8 - 20	5 - 17
No. 100 (150 μm)	2 - 10	2 - 10	2 - 10

Table 2: Typical Remedial Shotcrete Mix Designs
Proportions at the Nozzle in kg/m^3

Material	Mix Description			
	Wet-Mix		Dry-Mix	
	Plain	Silica Fume	Plain	Silica Fume
Portland Cement, Type 10	400	350	425	375
Silica Fume	-	47	-	50
10 mm Coarse Aggregate*	460	485	495	490
Concrete Sand*	1260	1215	1215	1205
Water	170	177	165	165
Water Reducing Admixture	Yes	Yes	-	-
Superplasticizer	-	Yes	-	-
Air Entraining Admixture	Yes	Yes	-	-
TOTAL:	2290	2274	2300	2285

*Proportions based on SSD Aggregates

Table 3: Properties of the Plastic Wet and Dry-Mix Shotcretes in Table 2

Property	Mix Description			
	Wet-Mix		Dry-Mix	
	Plain	Silica Fume	Plain	Silica Fume
Slump, mm	50	50	-	-
Air Content, %				
Before Shooting	8.5	6.4	-	-
After Shooting	4.8	3.9	-	-
Thickness to Sloughing, mm				
Overhead Application	80	130	65	380
Vertical Application	300	330	200	460
Overhead Rebound, %	15	13	46	20
Vertical Rebound, %	4	3	42	20

Table 4: Properties of the Hardened Wet and Dry-Mix Shotcretes in Table 2

Property	ASTM Test Procedure	Mix Description			
		Wet-Mix		Dry-Mix	
		Plain	Silica Fume	Plain	Silica Fume
Compressive Strength, MPa at 24 hours	C39	15	22	30	34
7 days		28	45	44	49
28 days		44	63	54	60
Flexural Strength, MPa at 7 days	C78	3.8	4.9	-	-
28 days		5.3	6.7	7.4	8.4
Boiled Absorption % at 28 days	C642	6.6	5.9	4.9	2.7
Volume of Permeable Voids, % at 28 days		14.3	12.9	11.2	6.3
Air Content, %	C457	5.0	3.3	4.3	3.9
Specific Surface, mm^{-1}		23	18	16	21
Spacing Factor, mm		0.25	0.36	0.35	0.26
Drying Shrinkage, Length Change, % at 64 days	C341	0.105	0.088	0.072	0.061

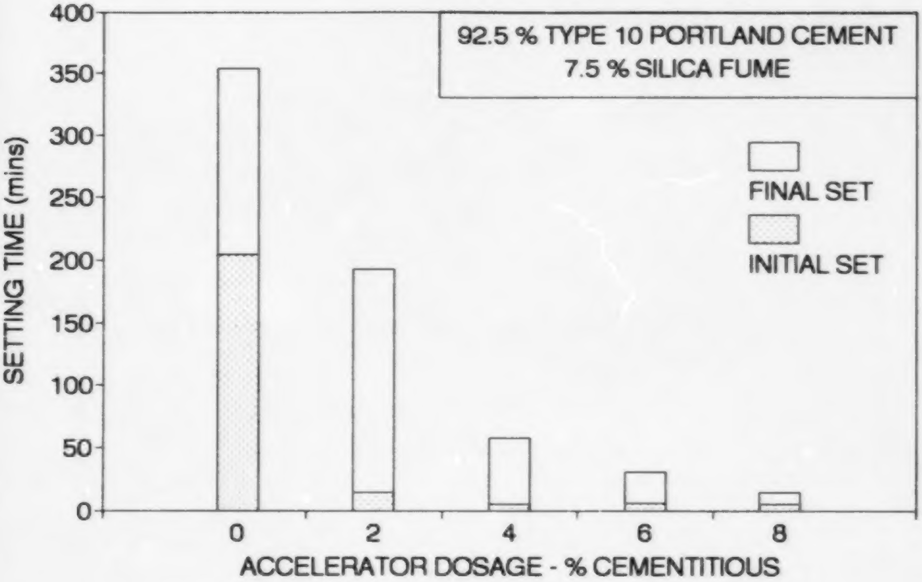

FIGURE 1: SETTING TIME TO ASTM C403

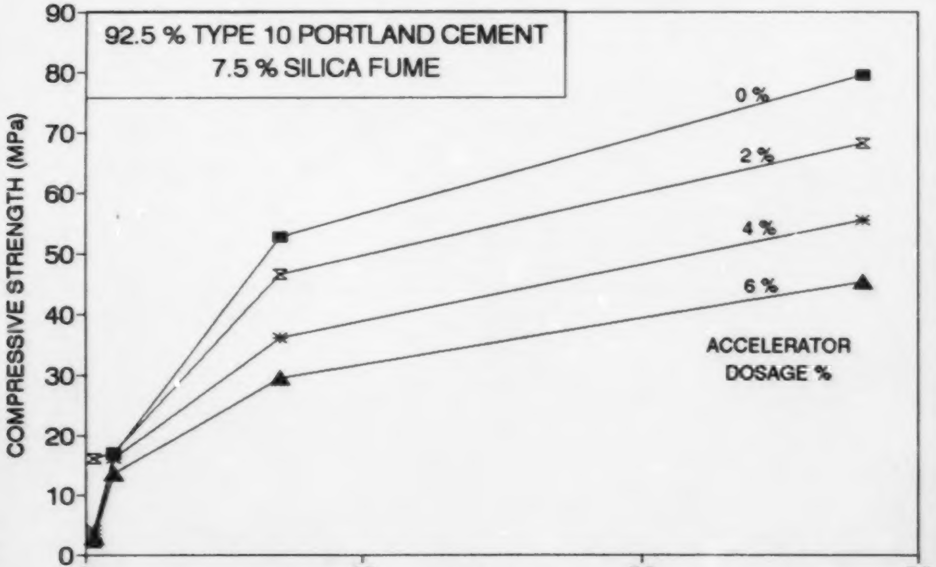

FIGURE 2: COMPRESSIVE STRENGTH OF 50 mm CUBES

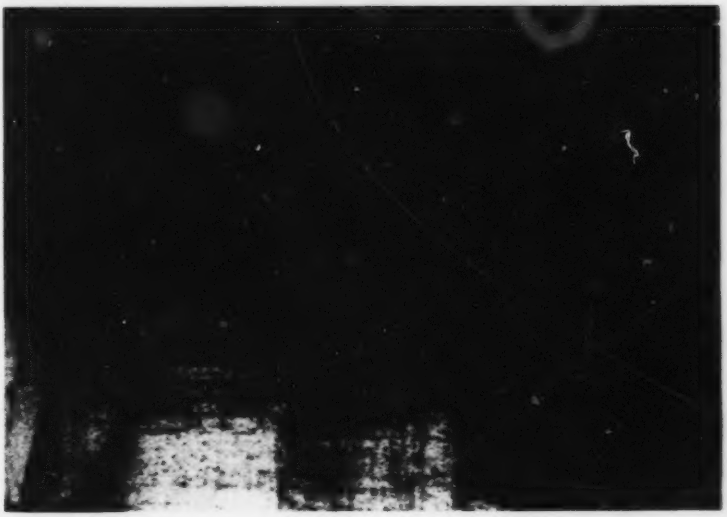

Figure 3: Restrained drying shrinkage cracking in a bridge abutment in Alberta repaired with an accelerated dry-mix shotcrete.

Figure 4: Shotcrete improperly applied to mesh reinforcement with build-up of shotcrete on the face of the mesh and shadowing and voids behind.

Figure 5: Stabilization of ravelling highway rock-cut in British Columbia, using high volume polypropylene fibre reinforced, wet-mix, silica fume shotcrete.

Figure 6: Rehabilitation of electrical transmission tower pier foundations in Skeena River Estuary, Northern British Columbia, using accelerated, dry-mix, steel fibre reinforced, silica fume shotcrete (6).

Figure 7: Bulk bin bag supply of steel fibre reinforced silica fume shotcrete for repair of the Jordan River Dam on Vancouver Island, British Columbia.

Figure 8: Barge load of dry-bagged premix shotcrete on its way from Vancouver, B.C. to a project near Alaska.

Figure 9: Mobile batcher unit supplying dry-mix shotcrete to a bridge rehabilitation contract in Montreal, Quebec.

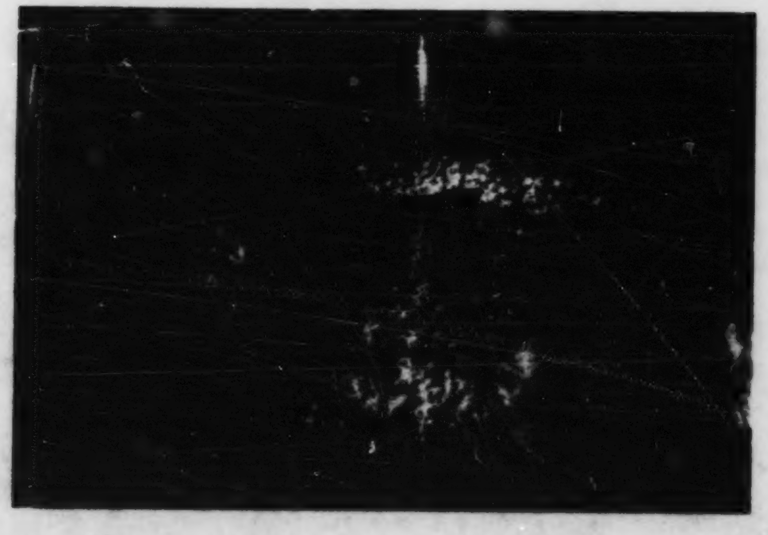

Figure 10: Wet-mix shotcrete repairs to berth faces in Port of Saint John, New Brunswick, completed in 1965, after 20 years in service. Shallow-pinned repair shotcrete peeled off berth face because of continuing deterioration of old concrete behind.

Figure 11: Nozzleman's helper removing overspray and rebound using an air-water lance during "top-down" shotcrete repair of the upstream face of the Jordan River Dam, Vancouver Island, B.C.

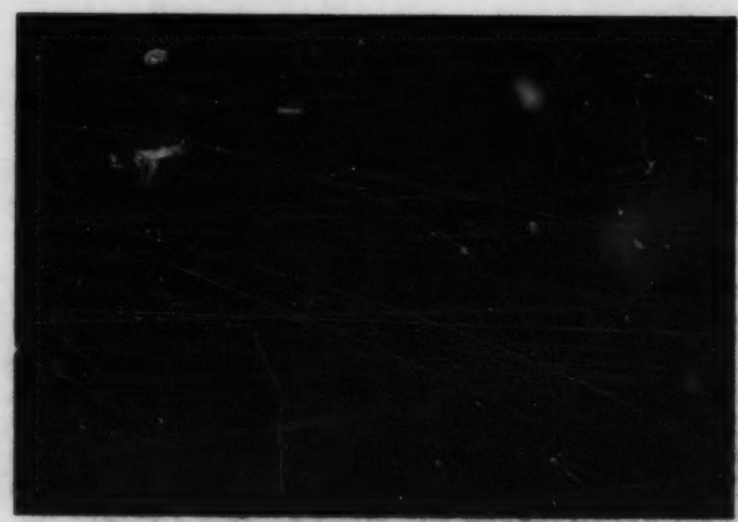

Figure 12: Vancouver Public Aquarium Beluga Whale Pool, artificial rockscape made with wet-mix silica fume shotcrete.

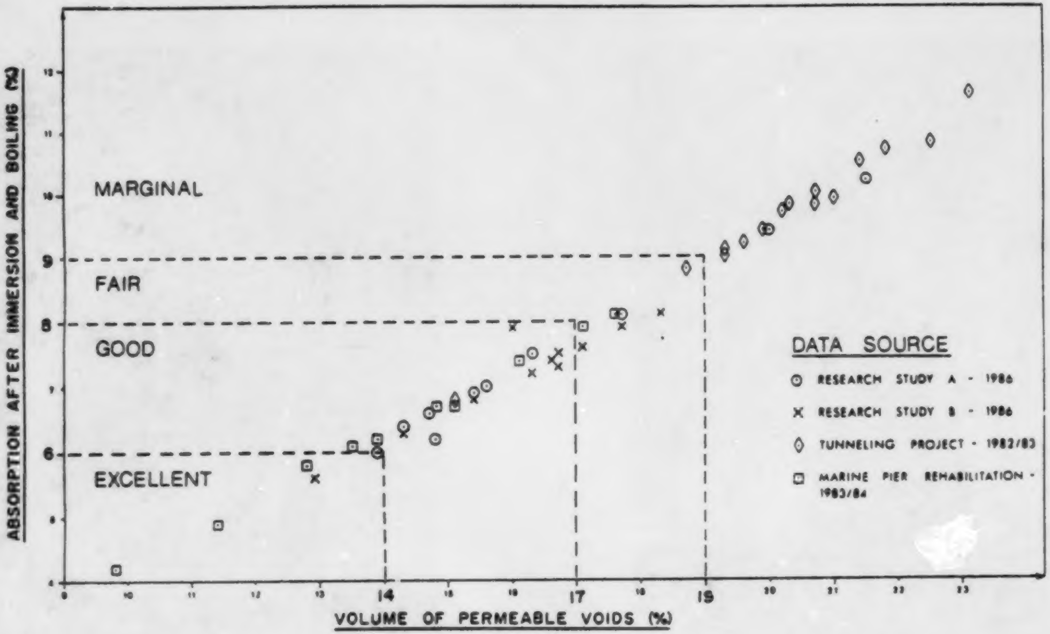

Figure 13: Relationship between absorption after immersion and boiling and volume of permeable voids of dry-mix shotcrete with suggested indicators of shotcrete quality (15).

Figure 14: Canada Place Trade and Convention Centre constructed on the shotcrete rehabilitated 60 year Pier B-C in Vancouver Harbour, B.C.

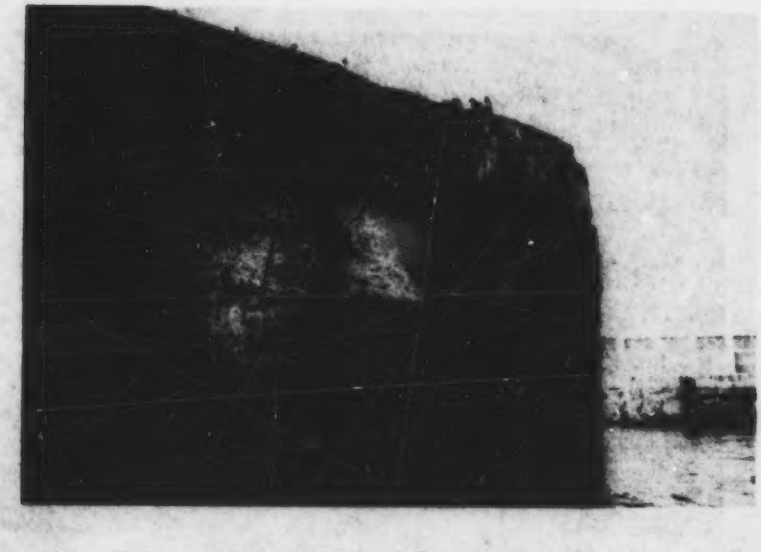

Figure 15: Deteriorated concrete in berth faces constructed in 1913 in the Port of Saint John, New Brunswick.

Figure 16: Repair of berth faces in Port of Saint John, New Brunswick, with air-entrained, wet-mix, steel-fibre reinforced, silica fume shotcrete (16).

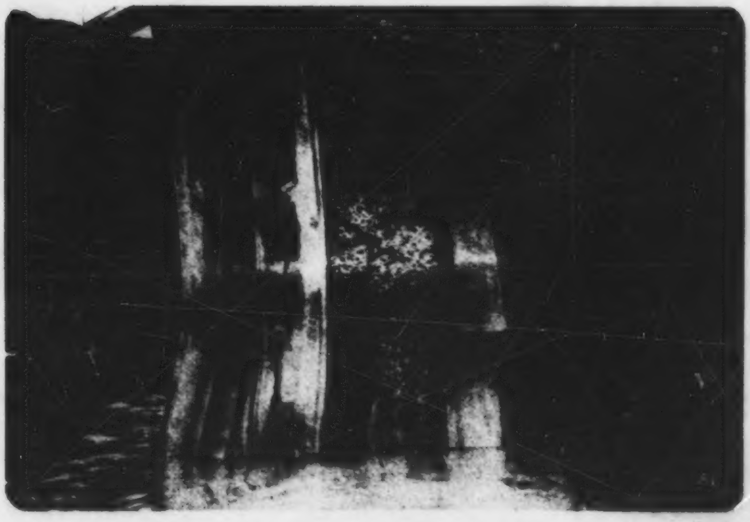

Figure 17: Deteriorated electrical transmission line piers in the Skeena River Estuary, Northern British Columbia.

Figure 18: Bridge piers repaired with dry-mix, steel fibre reinforced, silica fume shotcrete in 1986. Bridge is over the North Saskatchewan River at Drayton Valley, Alberta.

Durability of Repaired Concrete Structures

By Michel Pigeon and François Saucier

Synopsis:

There is a very fundamental distinction between the repair of a concrete structure and its partial reconstruction. The durability of the bonding between the old and the new concrete is a major parameter influencing the durability of a repaired structure, but not that of a partially reconstructed structure. A large number of tests were carried out to investigate the durability of the bonding between new and old concrete. The mechanical resistance of composite specimens (made of "old" concrete covered with "new" concrete) submitted to freezing and thawing cycles, wetting and drying cycles, simple air drying, as well as to a combination of these ageing treatments, was studied. Although the durability of the bonding between new and old concrete was found to be influenced by many parameters, the key role of differential shrinkage was confirmed. Microstructure studies of the interface between new and old concrete were also carried out. They indicate that the interface can be influenced by the type of cement used in the repair concrete or by the saturation state of the old concrete, and it was found that the durability of the bonding between new and old concrete could be related to the microstructural characteristics of this interface. In the last part of the paper, the results of slow tensile tests are reported. They show that the stress relaxation capacity of concrete in tension (a very important characteristic considering the key role of differential shrinkage in the build up of stresses at the interface) can be very significant, that it varies with the characteristics of the repair concrete, and that it can thus probably be improved. This possible improvement indicates in which way future research should be oriented to increase the durability of repaired concrete structures.

Keywords: concrete repairs, bonding durability, microstructure, differential shrinkage, slow tensile test, creep, fibers, air-entrainment

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INTRODUCTION

Let us first make a fundamental distinction between the repair of a concrete structure and its partial reconstruction. When a concrete structure shows signs of deterioration but is still basically sound, the "repair" of the damaged parts should normally restore the usefulness of the structure. But when, for instance, the removal of the damaged concrete requires temporary supports, or when new rebars and anchorages are placed in the repaired parts, this process should definitely be called a "partial reconstruction" (Fig. 1). Repairs cost less than partial reconstructions, but designers often choose the latter because they cannot be sure that the repair concrete will not debond with time. In fact, even if the repair of concrete structures is a rapidly growing field of concrete technology, optimal use of the materials and techniques is still far away, and how long repairs will last is very difficult to determine.

The durability of a repaired structure is much more than the durability of the repair material. In this paper, it is not the durability of the various repair materials which will be discussed (the durability of concrete has been the subject of a very large number of publications in recent years), but the durability of the composite structure made of an old concrete structure covered with a thin (25 to 100 mm) coat of "new" concrete. Will this composite structure be able to withstand the mechanical stresses, the thermal stresses, the effects of frost and of wetting and drying cycles, the effect of shrinkage of the "new" material pasted on the old and already shrunk concrete, without any significant deterioration?

If the repair material (the "new" concrete) is durable and the old concrete well protected, what type of deterioration is likely to occur?

The answer to this question is known by most engineers: it is the debonding of the "new" concrete at or near the bonding plane (that is often the weakest point of the composite structure). In fact, the interfacial zone is not only the preferential zone of rupture, but it is also the most stressed zone of the composite structure and it has a composition different than that of the two materials that are linked. The durability of the bonding is thus very different from the durability of both the old and the "new" concretes.

The problem, but not the answer, is therefore quite clear. The bonding mechanisms between "new" and old concrete need to be investigated in depth. This means fundamental research, similar to that carried out for the understanding of the characteristics of the aggregate-paste interface. Knowledge of the mechanisms which are responsible for the various stress build-ups at the interface (such as shrinkage, or compatibility problems) is required. The mechanisms related to the weathering of this bond (will it deteriorate with frost over the years, with wetting and drying cycles, etc?) must also be understood.

What do we know? What is in the technical literature? What is in the codes of practice? Many results on the quality of the immediate bonding obtained with various materials and techniques have been reported, but data on the durability of these repair systems is extremely scarce. The technical literature and the codes of practice contain important contradictions that cannot be clarified due to the absence of reliable laboratory and field procedures and data. For instance, a debate on whether the old concrete surface should be or should not be wetted before casting the new concrete would certainly be long and heated. Every engineer has his own ideas and experiences, but the influence of too many parameters is not well documented. It is thus impossible to put all the available information together and obtain a clear understanding of the phenomena involved.

This paper summarizes the results of a research program which started a few years ago and is still on going. Although this program addresses a lot of the questions related to the durability of repaired concrete structures, it has not yet in any way given the final answer to the problem (and, like any other concrete research program, it is doubtful that it will!). The available results do, however, answer some of the fundamental and practical questions, and point out in which way future research should be carried out to improve both our knowledge of the problem and the possibilities of improving the durability of repaired concrete structures.

There are three parts to this paper. In the first part, some of the mains findings of a series of tests carried out to analyze the influence of basic parameters (such as the wetting of the old concrete surface) on the durability of the bonding between "new" and old concrete are described. Composites specimens made of old concrete covered with "new" concrete were submitted to freezing and thawing cycles, to wetting and drying cycles, to simple air drying, and to a combination of these ageing treatments. In the second part, the question of the microstructure of the interface is discussed and the importance of this question is illustrated by the results of a series of tests carried out to analyze two of the parameters susceptible of having a significant influence. In the third and last part, the concept of total deformability is developed. A certain number of tests have already been carried out on this subject and are described. These test were carried out to try to answer a simple question: how can the stresses at the interface be reduced to a minimum? Because it is probably much better to reduce the stresses than to increase the strength. If the bonding is stronger than any of the two materials that are linked, fracture will simply occur just next to the interface in the weakest or most stressed of the two materials.

BONDING DURABILITY

As mentioned earlier, most of the test results that are available in the literature concern the quality of the bonding at early ages, i.-e. 1, 7 or 28 days. For instance, in the fifties, Felt (1) carried out a very comprehensive test program on thin bonded concrete overlays. He studied many different techniques of concrete removal and surface cleaning as well as many bonding agents and curing treatments, both in the laboratory and in the field. Hundreds of specimens were fabricated and tested, in most cases after 28 days. After considerable work, Felt concluded: "***it became apparent that factors influencing bond of new and old concrete were not easily isolated and controlled...***". No mention of bonding durability was made. Results of spot tests performed on field repairs after some years of service can be also found in the literature, but the lack of precise data on the conditions of exposure reduces the value of these answers.

To obtain data on new-to-old concrete bonding durability, simple procedures were used. New concrete (with a water to cement ratio of 0,40) was cast on an old concrete and, after 7 days of wet curing, these composite specimens were submitted either to daily freezing and thawing cycles, to wetting and drying cycles, to simple air drying, and, in some cases, to a combination of these. Periodical destructive tests were carried out to evaluate the evolution of the bonding strength with time. In this program, the strength of the bonding was not used as the performance criterion, since it is the evolution of the bonding strength with time that was considered important. A joint of moderate but stable strength is more durable that a stronger joint that shows a strength loss with time (see Fig. 2). Many different series of specimens were submitted at the same time to the same ageing treatments to allow valuable comparisons to be made [2].

The choice of the test method to measure the bonding strength was the first problem to solve, since no standard test is very widely used. For the purpose of this test program, the size of the specimens was a major parameter, because of the limited available spaces in the ageing devices. The time required to carry out a test was also considered since the fabrication of thousands of specimens was planned. To meet these needs, a special compression-shear device was designed. This apparatus applies a shear stress at the interfacial level of 75 mm cubic composite specimens submitted to a normal stress (see Fig. 3). The apparatus and the exact reasons for this choice of the testing method are described in a previous publication [3].

The parameters studied were chosen directly from basic field practices such as, obviously, the surface preparation of the old concrete (wet or dry), the use of cement pastes of various water-cement ratios as bonding agents, the time lag between the application of the cement paste as a bonding agent and the casting of the new concrete, and the use of silica fume as a bonding aid. At the time of writing, more that 3000 specimens had been tested and the program is still on going. This large number of specimens can be explained by the use of a set of at least 6 specimens for each single test, since dispersion of shear or tensile test results is high, and also because obtaining information on the evolution of strength with time requires testing of a control set of specimens immediately after curing and then testing others at various time intervals (typically 3) from there on. Ageing times varied from 1 day up to more than 600 days in some cases.

The first significant result from this test program is that, when good quality concrete is cast on a sound concrete surface to be repaired, the wetting of the old concrete does not increase the strength of the bonding. Fig. 4 compares the bonding strength obtained (at 7 days) for similar composite specimens fabricated twice: once on a dry surface and a second time on a surface wetted for one hour before casting the new concrete. A statistical analysis of the 87 pairs of results shown in Fig. 4 indicates that, globally, the saturation state of the old surface has no significant influence. However, when the results are separated in different groups, certain trends can be noted. In fact, wetting the surface for one hour prior to the casting of the new concrete reduced the bonding strength of the joints fabricated without any bonding agent or with a cement slurry at a water-cement ratio of 0,30 as a bonding agent. On the other hand, joints fabricated with a cement slurry at a water-cement ratio of 0,60 showed a slightly better mechanical strength when they were fabricated on a wet surface.

This comparison (Fig. 4) of the mechanical strength of composite specimens made of "new" concrete cast on wet or dry old concrete does not indicate if the saturation state of the old concrete has an influence on the durability of the bonding. To obtain this information, the evolution of the mechanical resistance with time must be compared for similar series of specimens fabricated on dry and wet surfaces. Fig. 5 gives examples of the evolution of the bonding strength with time for different types of specimens submitted to wetting and drying cycles. In this Fig. the results for six types of specimens are shown: three made with a cement slurry at a water-cement ratio of 0,30 as a bonding agent (one on a dry surface, another on a surface wetted for 1 hour, and a last one on a surface wetted for 24 hours), and three made with a cement slurry at a water-cement ratio of 0,60 (on similar surfaces). It can be seen in Fig. 5 that there is a decrease of mechanical strength with time for only one type of specimens, those made with a cement slurry at a water-cement ratio of 0,60 cast on a dry surface.

From the results in Fig. 5, obtained with specimens submitted to only one ageing treatment for a certain number of months, the conclusion would be that wetting the old concrete surface does not influence the durability of joints fabricated with low water-cement ratio pastes (or mortars) as bonding agents, but improves the durability of joints fabricated with high water-cement ratio pastes as bonding agents. This conclusion is in contradiction with other results obtained with specimens submitted to a combination of ageing treatments.

Fig. 6 shows the evolution of the bonding strength for similar series of specimens cast on wet and dry surfaces (with no bonding agent) and submitted to freezing and thawing cycles or wetting and drying cycles for almost a year and then exposed to simple air drying. At the end of the initial curing period (at 0 days), the specimens cast on a wet base show a lower bonding strength. However, since the evolution of the bonding strength (after a full year of accelerated ageing) is similar, it could have been concluded that the durability of the bonding is not influenced by the saturation state of the surface. However, after exposure to an additional 125 days of simple air drying, a significant difference clearly appeared: the specimens initially cast on a wet surface showed a larger loss of bonding strength than the specimens cast on a dry surface. Fig. 6 shows also the results for similar specimens air dried for the whole period. This single ageing treatment would not have not allowed to highlight the difference due to the wetting of the base concrete, since the two series of specimens show similar strength decreases.

The results in Fig. 6 clearly demonstrate that the influence of simple conditions, such as the wetting of the old concrete prior to casting the "new" concrete, is not very easily put in evidence. In the test program, combinations of ageing treatments have allowed to draw conclusions that would have not been obtained from any of the three simple ageing treatments (freezing and thawing cycles, wetting and drying cycles, and simple air drying). This underlines the complexity of the behavior of composite structures, and the importance of using adequate programs to correctly evaluate the potential durability of new-to-old concrete bondings.

The influence of the wetting of the old concrete has also been investigated in another way. Since it is a common field problem, several series of specimens were fabricated by exposing the bonding agent to partial drying before casting the "new" concrete. Cement paste with a water-cement ratio of 0,30 was brushed on a wet or a dry surface and then allowed to dry 45 minutes before casting the "new" concrete. To increase the effect of drying, an additional series of specimens was prepared with a paste brushed on a dry warm surface and allowed to dry 45 minutes in a warm room (37°C, 45% r.h.). The results in Fig. 7 show that moderate drying of the cement slurry brushed on a wet surface did not reduce the initial bonding strength, but did reduce it for the pastes brushed on a dry surface. Visual examination of the cement paste layer at the time of casting the "new" concrete clearly showed that the wetted surface had helped to delay the drying of the bonding agent (when compared to the paste brushed on the dry surface). It can be thus concluded that this delay in moisture loss allowed to maintain the qualities of the cement slurry bonding agent.

However, from these series of specimens made with partially dried cement slurry bonding agents, the most interesting results are the bonding strengths measured after almost 400 days of wetting and drying cycles. It can be clearly seen in Fig. 7 that the cycles allowed the progress of hydration, and this resulted in a gain in strength for the specimens made with cement pastes brushed on wet or dry surfaces and exposed to room air for 45 minutes. However, the joints cast on a cement paste exposed to warm dry air (38°C, 45% r.h.) for the same time did not show any gain in strength. Moderate drying of a cement slurry bonding agent can thus be "healed" with time and moisture, but a more severe drying will result in unrecoverable damage.

The lesson of the tests on the influence of a moderate drying of a cement paste used as a bonding agent is that the wetting of the surface can help to maintain the properties of the bonding agent for a certain time. However, other tests have shown (see Fig. 6) that the wetting of the old concrete surface could be detrimental to the bonding durability. Once again, this illustrates the difficulty of determining whether or not a given test procedure for bonding durability should be recommended. As a last comment on this topic, it can be pointed out that the "dry" surfaces used in the tests correspond to laboratory dry concrete, i.e. at about 50% r.h. In the field, after a few days of warm and sunny weather, a concrete surface is more likely to be "bone dry" and this could be a more severe condition than a "laboratory dry" surface.

The second important result from the various test series that were carried out concern the use of cement pastes as bonding agents. Some codes of practice allow the use of cement pastes with a water-cement ratio as high as 0,62 as bonding agents for thin concrete bonded overlays [4]. It was thus found of interest to evaluate the influence of the water-cement ratio of the bonding agent. As could be expected, 0,30 water-cement ratio cement pastes were generally found to perform better than 0,60 water-cement ratio pastes. Not only is the bonding strength higher with 0,30 water-cement ratio pastes, but durability is good. Cement pastes with a water-cement ratio of 0,60 give weaker joints that generally get weaker with time. This is not always the case as can be seen in Fig. 5, since the same bonding strength was obtained for 0,30 or 0,60 water-cement ratio pastes cast on a concrete surface wetted for 24 hours prior to casting the "new" concrete. It should also be pointed out that the joints fabricated without any bonding agent generally ranked between these two categories; these joints are stronger and maintain their strength better than those fabricated with a 0,60 water-cement ratio bonding agent, but 0,30 water-cement ratio bonding agents are still better.

The influence of the water-cement ratio of the bonding agent also appeared in the mode of rupture of the joints. In Fig. 8, the percentage of adhesion (evaluated after the rupture of the specimens) as a function of the time of exposure to wetting and drying cycles is shown for the same series of specimens as in Fig. 5. From Fig. 8, it is clear that, even if the bond strength is similar in many cases (see Fig. 5), the fracture mode is not. For the cement pastes of higher water-cement ratio, fracture tends to propagate directly at the interface between the base concrete and the "new" material as illustrated in Fig. 9. For the lower water-cement ratio pastes, fracture generally propagated in the bulk concrete or at the contact between the cement paste and the "new" concrete cast on it, exposing mostly aggregates. This last type of fracture was often obtained, which underlines that the absence of aggregate interlocking between the "new" and the old concrete creates a plane of weakness, even if a strong bonding agent is used.

A low water-cement ratio cement paste is unfortunately not the final answer to the quest for an efficient bonding agent. Fig. 10 shows the bonding strength evolution of different joints (all made with a 0,30 water-cement ratio paste) submitted to almost 400 days of wetting and drying cycles, and then submitted to daily freezing and thawing cycles for 112 additional days. In this specific case, only the strongest joint (one of the strongest tested in the complete project) showed a strength decrease due to freezing and thawing cycles. This can be attributed to a good hydration of the bonding agent during the wetting and drying cycles (more than one year) resulting in a very dense layer between the "new" and the old concrete. When freezing occurs, such a dense layer can cause stress build-ups at the interface due to restrictions to water movements. The cement paste bonding agents of the others joints which results are shown in Fig. 10 were cast on dry surfaces which partly explains their lower strength. The conclusion to draw from Fig. 10 is that the water-cement ratio of the cement paste used as a bonding agent should not be very different from that of the concretes being bonded. Once again, if only the results after 400 days of a single ageing treatment had been considered, this conclusion could not have been drawn.

The third main result from the research project is the identification of the relative influence of the three ageing treatments used. Globally, simple air drying was found to be the main cause of deterioration. At the opposite, freezing and thawing cycles mostly had a positive effect, probably due to hydration during the tests, even if this ageing treatment can reduce the bonding strength in certain cases (see Fig. 10). Wetting and drying cycles gave intermediate results, being detrimental mainly for the joints fabricated with a 0,60 water-cement ratio paste. As already shown, exposure to a combination of ageing treatments gave results that were very difficult to predict and thus unforeseeable in many circumstances. This clearly shows the complexity of the problem and the difficulties involved in trying to solve it.

The ranking of the detrimental influence of the three ageing treatments can probably be simply related to the amount of drying shrinkage. The specimens submitted to freezing and thawing cycles were always kept saturated; no drying shrinkage was therefore involved and only a few of the joints were damaged during exposure to 100 daily cycles. During the wetting and drying cycles, drying periods of 2 or 3 days alternated with wetting periods of 2 or 3 days, thus allowing some shrinkage to occur. Poor quality joints were weakened by this moderate shrinkage. Clearly, the wetting periods allowed the progress of hydration for many specimens, resulting in a higher bonding strength. Simple air drying allowed maximum drying shrinkage to occur and this was detrimental to almost all joints.

It is known that differential shrinkage between already shrunk old concrete and "new" concrete cast on it tends to cause stresses at the interface and can result in debonding. The results of the tests confirm this fact, but also indicate that other mechanisms of deterioration, such as the damaging of high strength joints by frost, are possible. The importance of shrinkage of course brings up the still unanswered question: "since drying shrinkage will necessarily occur, how can the stresses be reduced?" The third part of this paper addresses this question.

Another important consideration as concerns the bonding durability is the fact that, in the field, the process used to remove the unsound concrete (hydrodemolition, jack hammer, scarifier, etc.) can damage the sound concrete that is left. This was not included in the tests since, in a recent publication, this problem was analyzed in detail [5].

MICROSTRUCTURE OF THE INTERFACE

When "new" concrete is cast against a surface of old concrete, special conditions exist at the interface which influence the structuring of this "new" concrete and of the cement paste. As shown in Fig. 11, this wall-effect exists both at the level of the aggregates and of the cement grains. At the level of aggregates, the absence of aggregate interlocking between the old and the "new" concrete simply results in a plane of mechanical weakness. At the level of the cement grains, the contact of a fresh cement paste with a hard surface results in a transition zone.

Many investigators have analyzed the transition zone between cement paste and aggregate in concrete [6]. It it most probable that the zone of transition that is formed when "new" concrete is cast against old concrete is quite similar. In fact, in such a case, as shown in Fig. 11, two different types of contact zones exist: "new" cement paste in contact with aggregates, or with hardened paste. Since the transition zone between paste and aggregate is well documented, the tests were focused on the transition zone between "new" cement paste and old hardened cement paste.

There are various way of studying the transition zone. The advantage of the X-ray diffraction technique (explained below) is that the composition of a surface of few centimeters square can be analyzed in a single operation, which is thousands of times larger than the surfaces observed with a scanning electron microscope.

The X-Ray diffraction technique, developed in France [7], is illustrated in Fig. 12. A composite sample is fractured at the interface, placed on a rotating support in the X-ray diffraction device, and the composition of the crystalline phases present on that plane is established by an X-ray diffraction analysis. By very light abrasions (about 10 μm of material removed each time), successive planes are exposed and analyzed by X-ray diffraction. The result is a profile of the crystalline composition from the interface to the bulk of the paste.

The main information obtained with this technique is the orientation index of the portlandite. The orientation index value is the actual ratio of the two major peaks of portlandite divided by the usual value of this ratio for portlandite powder (0,74, see Fig. 13). When the portlandite present at the interface being analyzed is similar to a powder (small grains, no preferential orientation), the value of the orientation index is 1 or nearly so. Higher values of the orientation index (3, 4, 10 or more) indicate that portlandite has crystallized in an "different way".

Although the exact significance of a high orientation index is still controversial, it is clear that different values of the orientation index indicate significant microstructural differences. High values of the orientation index are obtained when the 101 plane of the portlandite is predominant on the 001 plane. It is not clear if this is mostly due to a preferential orientation of the portlandite, or simply to the presence of large crystals. However, as illustrated in Fig. 14, preferentially oriented or large crystals can only form if large spaces are available. High values of the orientation index can thus most probably be considered as an indication of a larger porosity at the time of portlandite crystallization (in the first hours after casting the paste), or at least of a significant variation in the microstructure.

It was shown in the first part of this paper that wetting the old concrete surface can sometimes have a detrimental influence on the durability of the bonding. Tests were thus carried out to analyze the possibility that the saturation state of the old concrete could influence the properties of the paste near the interface (it was not considered likely that the saturation state of the old concrete could influence the properties of the paste away from the interface) [8]. The measurement of the orientation index is particularly well suited for this type of study (i.e. of a relatively large zone but of limited thickness).

The profile of the orientation index was studied for four types of joints (i.e. fabricated with two Portland cement pastes cast on either wet or dry supports of one year old concrete). In order to analyze the properties of the interface between "new" and old paste only, the "new" pastes were cast on the moulded surfaces of concrete specimens lightly lapped to removed the first 1 or 2 mm of laitance. After a 24 hour curing period, the joints were fractured at the interface (or nearly so), and the profile of the orientation index was determined.

It was found that not only the saturation state of the support, but also the nature of the cement, could have a great influence. Fig.s 15 and 16 show the profiles of the orientation index for both cements and both saturation states of the old concrete. For cement A (Fig. 15), the orientation index is quite high at the interface and decreases to normal values at about 20 or 30 µm from it. As can be seen in this Fig., the orientation index rises to a higher value at the interface for the paste cast on a wet support, indicating that this can have an influence on the microstructure. At the opposite, the joints fabricated with cement B did not show any high values of the orientation index close to the interface (Fig. 16). This was subsequently explained by the faster rate of solubility of the sulfates in this cement which allowed the rapid formation of ettringite restraining the formation of large portlandite crystals.

The conclusion of these tests is then twofold: first, the wetting of the old surface can influence the microstructure near the interface, and, second, depending on the cement characteristics, other phenomena can be superimposed on these microstructure changes. It must be emphasized that this does not mean that the wetting of the old concrete has not influenced the transition zone of the joints made with cement B, but that, due to certain properties of the cement, the orientation index values remained close to 1 at the interface. In fact, all composite specimens studied in the tests reported in the first part of this paper were fabricated with cement B, and it was shown that wetting the old concrete surface could have a detrimental influence on the bonding durability.

The influence of the wetting of the old concrete surface can probably be explained by its influence on the water exchanges between the fresh paste and the old material. Working on the adhesion of cement paste to clay brick, Grandet [9] demonstrated by X-Ray diffraction and mechanical tests that good quality bonding was obtained when a moderate flow of water from the paste to the porous base took place. Fig. 17 describes three cases: 1 - when the clay brick is totally dry, the water is rapidly sucked out from the paste, hydration is poor, and bonding is very poor; 2 - when the clay brick is partially saturated, a certain amount of water is gently absorbed by the support, and this increases the penetration of ettringite in the pores of the base material resulting in a good quality bonding; 3 - when the clay brick is saturated with water, no water exchanges take place, ettringite crystals do not tend to penetrate the porous base and the bonding is lower than on a partially saturated base.

The laboratory results obtained on porous clay brick can probably be applied to concrete, even if the porosity of concrete is quite smaller than that of clay brick. The porosity of old concrete to be repaired can vary in a very wide range, depending on the water-cement ratio of the mixture and its ageing conditions. It is thus impossible to determine an optimum saturation level to enhance the development of a good bonding. It seems reasonable, however, to at least conclude that bone dry or totally wet concretes are not the best. The water-cement ratio of the "new" concrete must also be considered, since the amount of available water is very different between a normal mixture and a high strength one. The overall result is not obvious.

The laboratory data presented in this part of the paper indicates that the microstructure of the interfacial zone between a "new" concrete and an old one can be influenced by very simple conditions, such as the saturation state of the old concrete. And if the microstructure can be affected, so can obviously the strength and durability of the bonding. Little data are available on this subject, and this is clearly an area where research is needed to allow a better understanding of the nature of the bonding between "new" and old concrete.

TOTAL DEFORMABILITY OF THE REPAIR CONCRETE

The most important problem to solve, as the results described in the first part of this paper tend to confirm, is the generation of stresses at the interface due to differential shrinkage between the old and already shrunk concrete and the "new" concrete (which slowly undergoes normal drying shrinkage). Increasing the bonding strength is not the best solution, because the strength is always limited by the weakest of the following parts: the "new" concrete, the bonding, and the old concrete (and old concrete is very often a relatively poor material). The answer is therefore not in very strong joints, but in the reduction of the stresses associated with shrinkage.

Reducing shrinkage in itself can be considered. Since shrinkage originates from the hydration products, it is possible to reduce the volume of the hydrates by using leaner mixes. Unfortunately, this is, in a way, self-defeating, because leaner mixes normally have higher water-cement ratios, and shrinkage increases with this ratio. Using high-strength concrete with a very low water-cement ratio and less shrinkage is not the answer either, because its higher modulus of elasticity will increase the stresses due to shrinkage deformation. The use of high-strength concrete will also cause compatibility problems due to its different mechanical properties and very low porosity (see Fig. 10).

The key word thus become "relaxation". Are there ways to increase the possibility of stress-relaxation in tension while shrinkage gradually increases the applied deformation and thus the stress? Obviously there is some creep in tension, but very little data are available on this subject. Another possible way of stress relaxation is "diffuse microcracking". This mechanism was reported as being efficient for thin cementitious wall coatings [10] where massive air entrainment was found to cause the formation of a dense and uniform network of submicrocraks without any "large microcracks" breaking the homogeneity of the lining. These two solutions (i.e. creep and diffuse microcracking) were investigated in the project described below.

Creep in compression must be considered in many situations: column design, deflection calculations, prestressed members, etc. A large number of publications have thus been written on this subject. Creep in tension, however, is poorly documented, probably because the tensile strength of concrete is low and not even considered in most codes of practice. What are the parameters that influence creep in tension, particularly when concrete is slowly drying and is not under the usual "no-loss-of-water" condition? Basic creep in compression is associated with water movements in the hydrates which are highly stressed by compression; can relatively low tensile stresses induce a similar behavior? Brooks and Neville [11] have made some measurements and observed, over a 90 day period, that the creep rate in tension stayed almost constant, whereas the same concrete in compresion showed the usual decreasing rate of creep with time. This evidence supports the idea of using concrete mixtures able to relax by creep a significant part of the stresses induced by differential shrinkage.

Stress relaxation by "diffuse microcracking" is another interesting possibility. It has long been accepted that deformation after the peak load is due to the opening and growth of microcracks. But microcracks also play a role before the peak load. Attigobe and Darwin [12] have calculated that the deformation before the peak load in compression due to the opening of microcracks is very significant. Ansari [13] has demonstrated, using laser interferometry, that microcracks appear long before the peak load in tension, and that the microcracked zone shows a deformation up to 10 times larger than that measured on the total length of the specimen. Diffuse microcracking is probably a function of the loading rate, since rapid loading does not allow cracks to stop on small obstacles. The question is then: "Can diffuse microcracking before the peak load be maximized and optimized to play a significant role in stress relaxation?"

From what was just mentioned, it can be said that damage to the repair concrete and loss of bond will occur only when the strain imposed by differential shrinkage becomes greater than the sum of the elastic strain, the creep strain, and the strain related to diffuse microcracking. The sum of these strains can be called the "**total deformability**" of the concrete. The relative importance of each of the three strains and the ways to promote a higher stress relaxation by creep or diffuse microcracking is not known. To obtain information on this subject, a research project was carried out to evaluate the practical possibility of producing concrete specifically designed for repairs.

Three possible ways to enhance the stress relaxation capacity of concrete were analysed: the use of latex, the use of air entrainment, and the use of steel fibers [14]. All three parameters are considered to modify the cracking behavior of concrete (see Fig. 18). Styrenebutadienne latex forms a co-matrix inside the capillary pore system, and small microcracks have to "tear up" this membrane to propagate. The air voids act at a higher level, since they can play the role of microcrack arresters by modifying the radius of curvature at the tip of microcracks. Steel fibers can also play the role of crack arresters, but at a higher level than the air voids, since they are larger and much less numerous.

To analyze these parameters, a special apparatus (described elsewere) was built [14]. In this apparatus, tensile specimens can be brought to failure in a few minutes or in about 12 hours. In the rapid tensile test, only a very small fraction of the creep and diffuse microcracking capacity can be observed. Slow tensile tests, however, allow a larger fraction of the creep and diffuse microcracking capacity to take place. As shown in Fig. 19, the capacity of the material to slowly relax stresses can be estimated by comparing the stress-strain curves obtained in rapid and in slow tensile tests. It is obvious that only a limited part of the creep and diffuse microcracking capacity can be observed in 12 hour tests, and that the total strain relaxation capacity is probably much larger. The 12 hour tensile test can thus be considered as an "accelerated" creep or relaxation test. The ideal test method would probably be a restrained shrinkage test in which shrinkage is prevented with the use of a rigid frame and the strain relaxation is evaluated by monitoring the stress build up with time. However, in this procedure, a given specimen can stay in the apparatus for days or weeks, and obtaining information thus either requires a lot of time or a lot of equipment.

In this series of tests, the three parameters were used in a 2k factorial experimental design. Eight mixtures were thus fabricated, using each possible combination of the three additions (latex, air entrainment, and fibers): the basic mixture with no addition, one with each of the three additions, one with latex and fibers, one with latex and air entrainment, one with air entrainment and fibers, and one with latex, air entrainment and fibers. There were thus four mixtures made with latex and four without, four with air entrainment and four without, and four with fibers and four without. The basic concrete mixture chosen had a water-cement ratio of 0,40, contained a 6 mm maximum size aggregate, and was water cured for 2 days. This type of mixture was used since it is representative of those used for thin bonded repairs.

The first significant result from this series of tests is that the deformation in tension is approximately 30% to 60% higher in slow tests (6 to 12 hours) than in rapid tests (2 to 4 minutes). Stress-relaxation in tension due to some form of creep ("normal" creep or diffuse microcracking) is thus possible and significant. The second one is that normal air entrainment seems to have little influence. The third one is that fibers can increase the deformation at failure in slow tensile tests, but only at early ages (i.e. before 28 days). The last one is that latex has little influence at early ages, but a very negative influence if the specimens are tested after 28 days of air curing.

Steel fibers are known to enhance significantly the strain capacity of concrete after the peak load. In this series of tests, however, the fibers were found to have a positive influence before the peak load. In the specimens tested immediately after curing (at 2 days) and in those tested after 16 days, the total strain measured in the slow tensile tests was 50% to 70% larger for the steel fiber concretes than for the non-fiber-reinforced concretes (see Fig. 20). This influence was not observed in the specimens tested after 28 days (i.e after a significant air drying period). It is probable that the relative ductility of the cementitious matrix at early ages allows some slipping of the fibers, but that this mechanism of strain relaxation becomes negligible with the hardening and drying of the matrix. It is very interesting to note that the positive influence of the fibers seems to occur when the shrinkage rate is maximum.

The behavior of the latex-modified concretes, a complete inhibition of creep in tension after 28 days of air curing, was not expected. As illustrated in Fig. 21, no significant difference between the latex-modified concretes and the non-latex-modified concretes was observed in the specimens tested at 2 days and at 16 days. After 28 days of air curing, however, the latex-modified concretes appeared to have lost all their relaxation capacity during slow tensile tests. This phenomenon, which was observed on the 4 different latex-modified concretes fabricated, could be explained by the sealing effect of the latex. Creep is associated to moisture movements, and, as the concrete dries, the latex coagulates and seals the capillary pores. At early ages, the concrete is still moist, the latex has not fully coagulated, and moisture movements are possible. After 28 days, coagulation has progressed significantly, and creep could be inhibited by the sealing effect. More tests are of course required to confirm and fully understand this phenomenon.

CONCLUSION

Are durable repairs possible? Felt [1] answered this question in 1956 when he demonstrated that thin bonded concrete overlays built at the beginning of the century were still performing well. The problem is that we still have not learned how to reproduce them systematically. Part of the solution probably lies in recognizing that repaired structures act as a composites and should be analysed as such. The discontinuity in properties between the "new" and the old concrete at the interface means much more than only a change in the mechanical capacity of the repaired members. And, regardless of the repair method that is selected, there will always be special problems at the interface, either due to the absence of aggregate interlocking or to microstructural phenomena.

As the test results presented in the first part of this paper indicate, bonding durability is a function of many parameters, and it would be difficult in the present state of the knowledge to develop a simple test procedure to measure bonding durability that would be applicable in all circumstances. It seems clear, however, that the bonding agents used should have properties as close as possible to that of the concretes being bonded. If a cement slurry is to be used as a bonding agent, its water-cement ratio should probably be in the 0,40 to 0,50 range. This slurry, or the repair concrete if no slurry is used, should probably not be placed on a very dry concrete surface, nor on a fully saturated surface, but preferably on a partially saturated surface (obtained by wetting the surface and letting it dry for a short period of time prior to casting the slurry or the repair concrete).

The key role of differential shrinkage has long been recognized, but solutions to this problem are still to be found. One of the possible ways is to optimize the stress relaxation capacity of the repair concretes. The limited number of tests carried out on this subject clearly show that concretes generally have a relaxation capacity in tension, but the key parameters are not known and must therefore be investigated. What is the influence of the water-cement ratio and of the Type of cement, etc.? Developments in this area could lead to made-to-measure concretes suited for various repair situations. The test results also indicate that diffuse microcracking is perhaps not the best way to substantially increase the stress relaxation capacity of concrete, but the use of steel or carbon microfibers, for instance, could improve this capacity significantly. It is clear that research in this field must be pursued, because it is the most promising way to increase bonding durability which is the major problem affecting repairs.

ACKNOWLEDGEMENTS

The authors are grateful to the Natural Sciences and Engineering Research Council of Canada and to Ciment St-Laurent, Montréal, Canada, for their financial support for this project.

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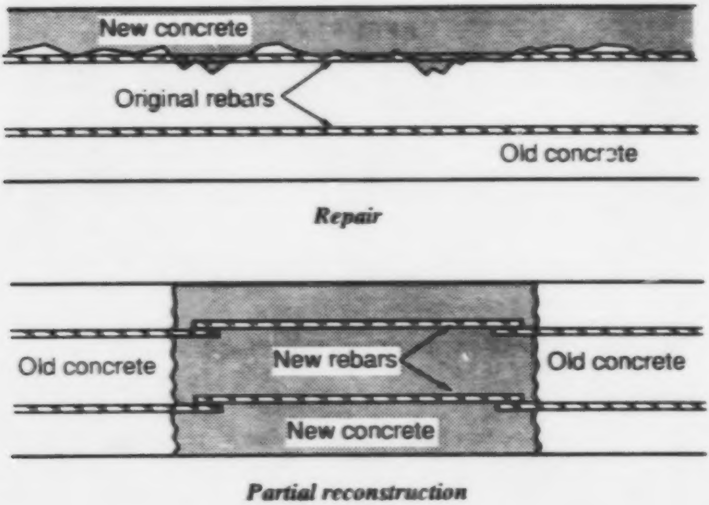

New concrete
Original rebars
Old concrete
Repair
Old concrete
New rebars
Old concrete
New concrete
Partial reconstruction

Fig. 1 - Repair versus partial reconstruction.

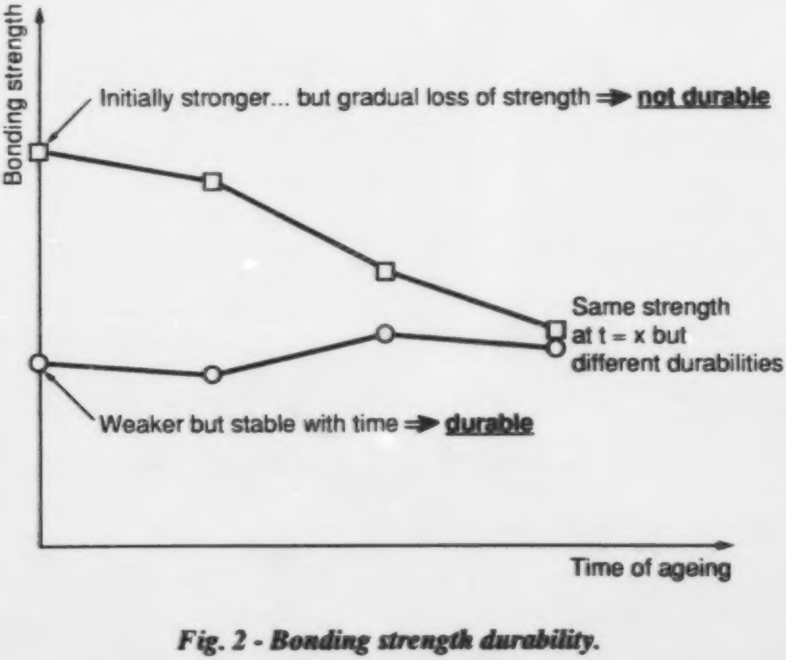


Fig. 2 - Bonding strength durability.

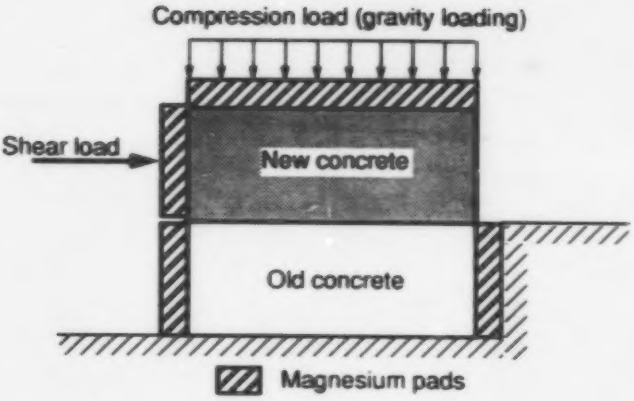

Compression load (gravity loading)
Shear load
New concrete
Old concrete
Magnesium pads

Fig. 3 - Shear strength measurement.

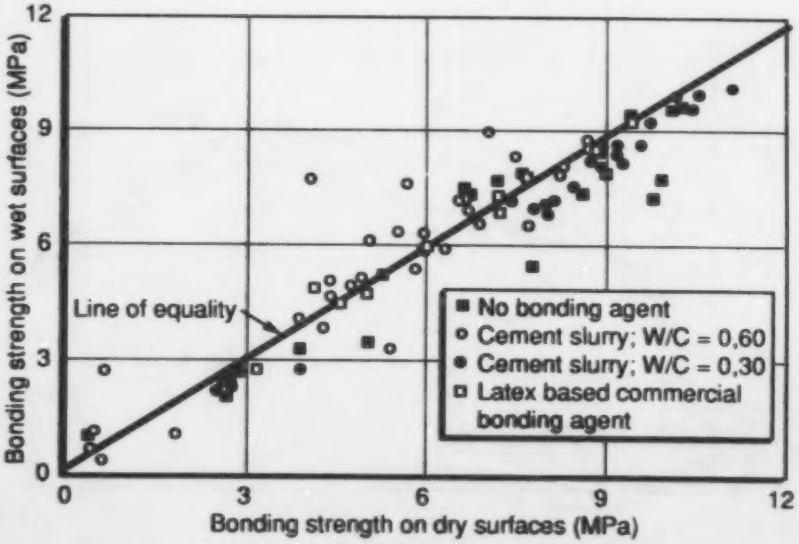


Fig. 4 - Bonding strength on dry and wet surfaces.

(from ref. 2)

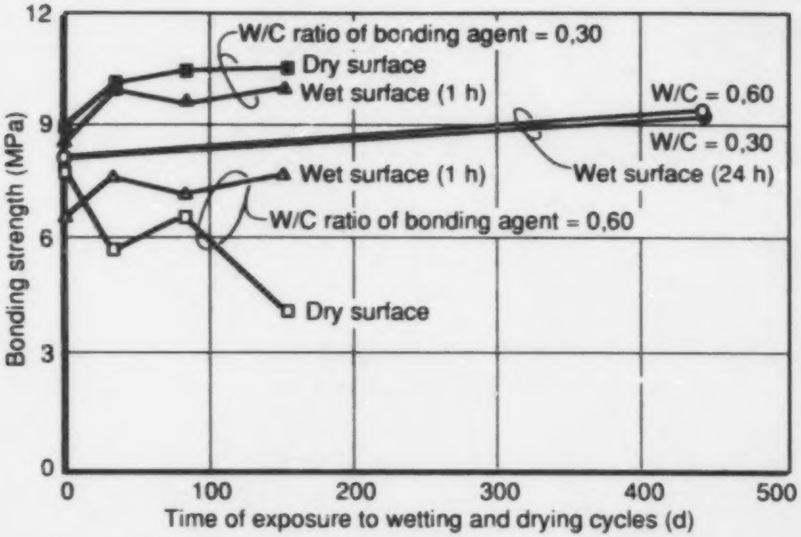


Fig. 5 - Bonding strength durability versus water-cement ratio.

(from ref. 2)

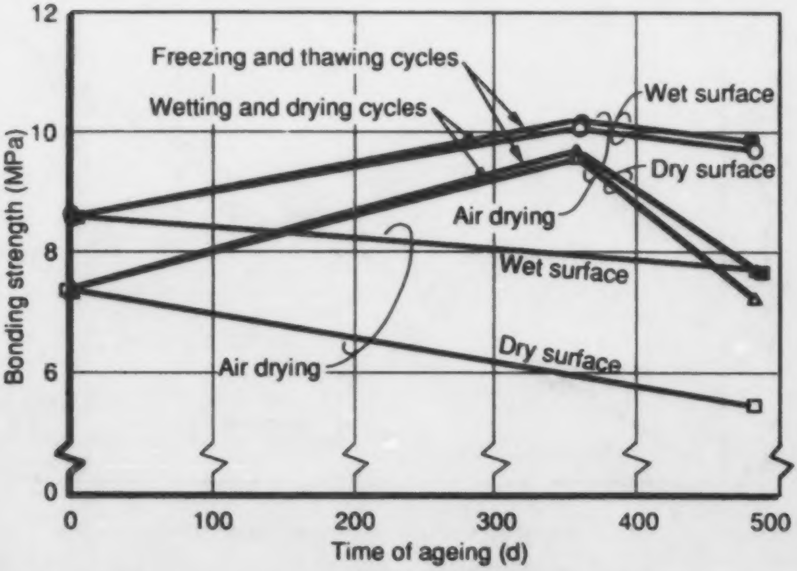


Fig. 6 - Influence of saturation state of support on bonding durability (combined ageing treatments vs simple air drying).

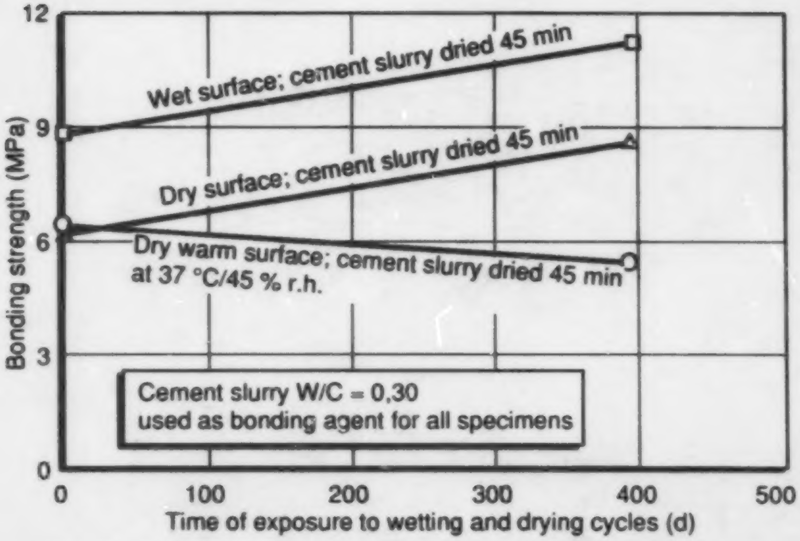


Fig. 7 - Influence of drying of cement paste bonding agent on bonding durability.

(from ref. 2)

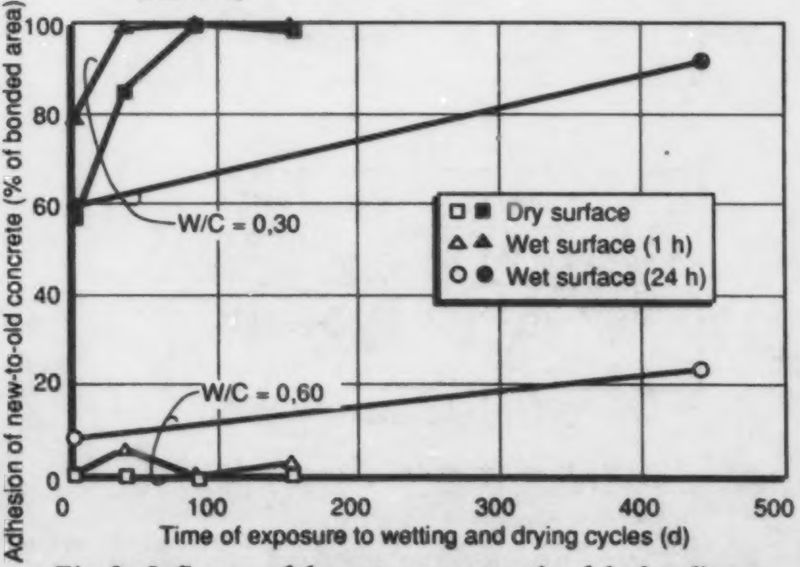
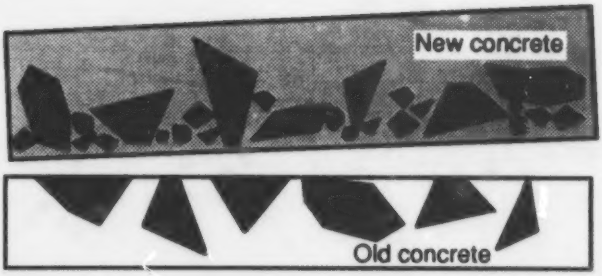
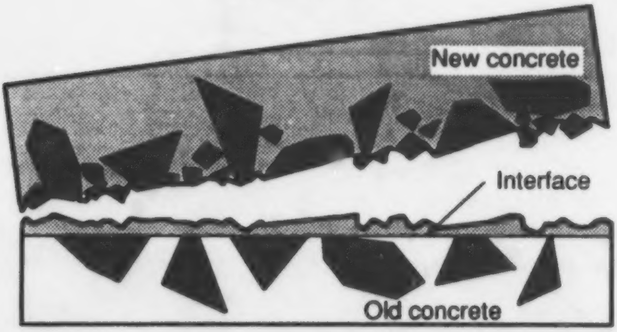


Fig. 8 - Influence of the water-cement ratio of the bonding agent on the percentage of adhesion.

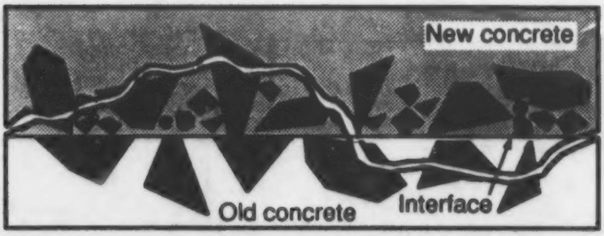
(from ref. 2)


New concrete
Old concrete

a) 0% adhesion.


New concrete
Interface
Old concrete

b) 100% adhesion.


New concrete
Old concrete
Interface

c) 100% adhesion.

Fig. 9 - Fracture modes of concrete specimens.

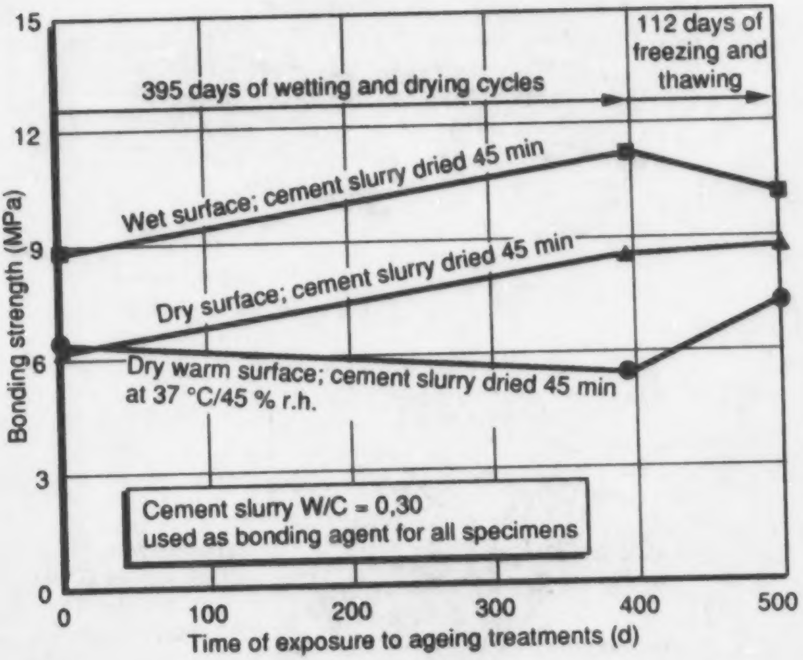


Fig. 10 - Influence of combined ageing treatments on bonding durability.
(from ref. 2)

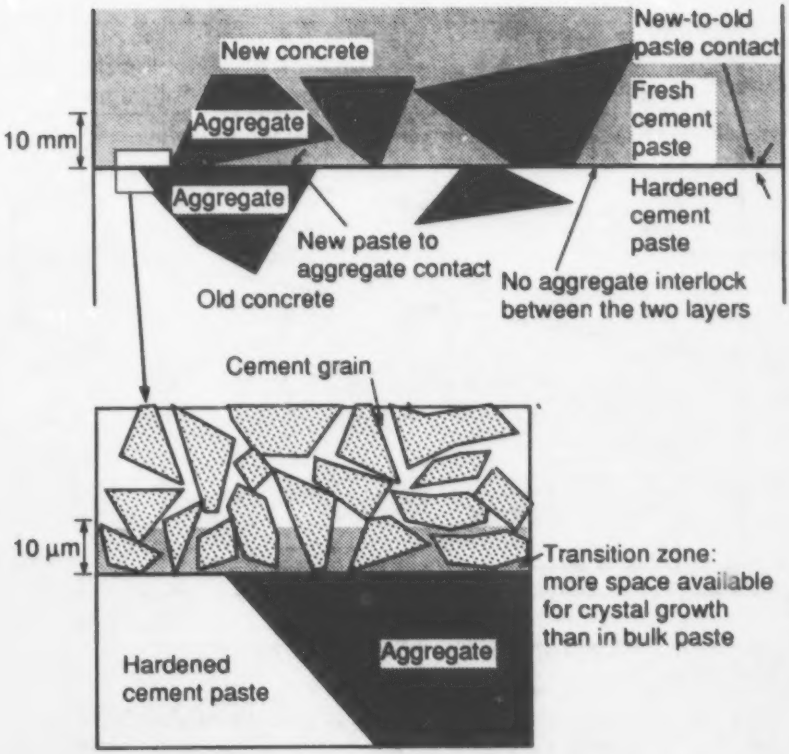

New concrete
New-to-old paste contact
Fresh cement paste
Aggregate
10 mm
Aggregate
Hardened cement paste
New paste to aggregate contact
Old concrete
No aggregate interlock between the two layers
Cement grain
10 µm
Transition zone: more space available for crystal growth than in bulk paste
Aggregate
Hardened cement paste

Fig. 11 - New concrete to old concrete contact.

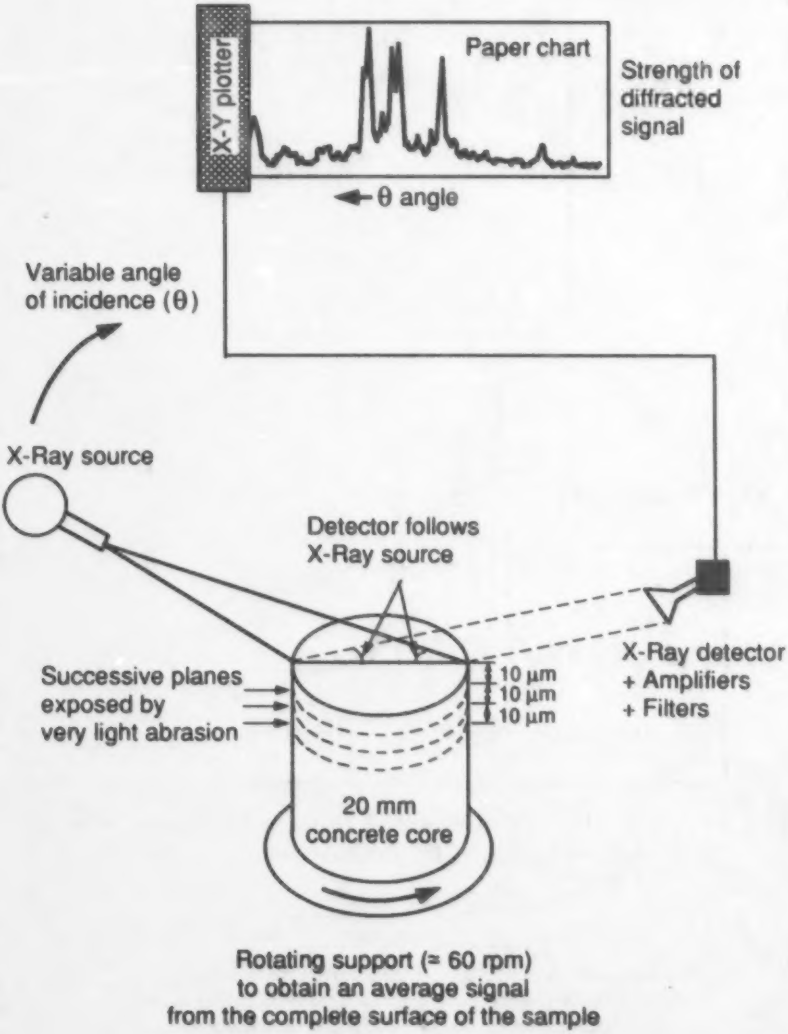
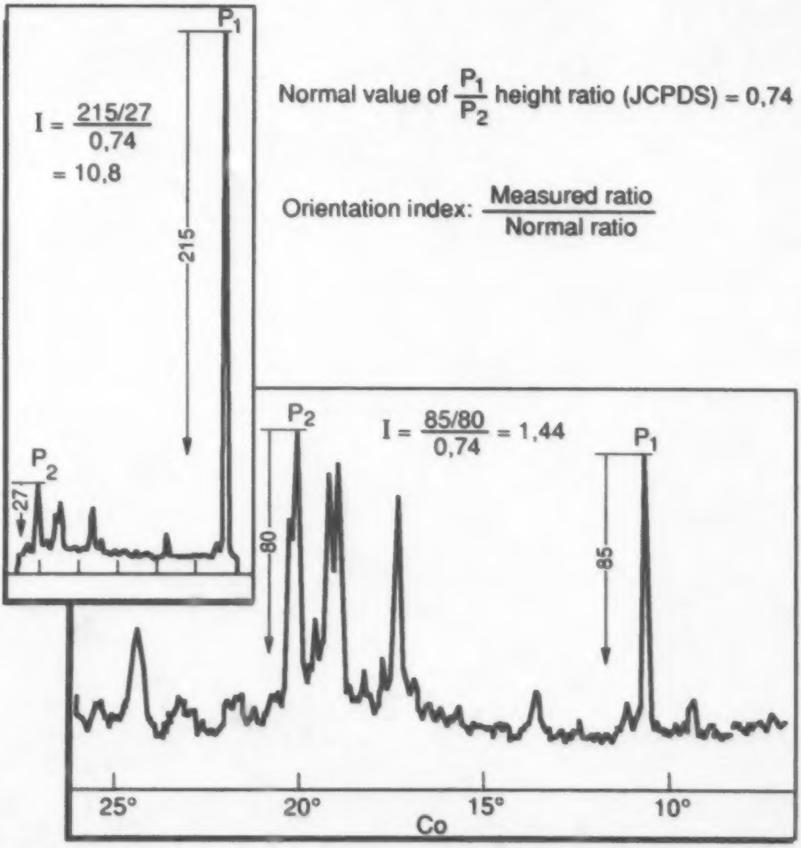

X-Y plotter
Paper chart
Strength of diffracted signal
θ angle
Variable angle of incidence (θ)
X-Ray source
Detector follows X-Ray source
X-Ray detector
+ Amplifiers
+ Filters
Successive planes exposed by very light abrasion
10 µm
10 µm
10 µm
20 mm concrete core
Rotating support (≈ 60 rpm) to obtain an average signal from the complete surface of the sample

Fig. 12 - X-Ray diffraction technique.



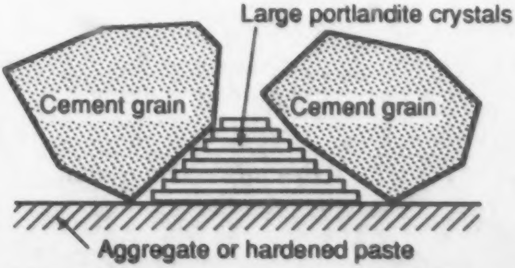
Normal value of $\frac{P_1}{P_2}$ height ratio (JCPDS) = 0,74

Orientation index: $\frac{\text{Measured ratio}}{\text{Normal ratio}}$

$I = \frac{215/27}{0,74} = 10,8$

$I = \frac{85/80}{0,74} = 1,44$

Fig. 13 - X-Ray diffraction diagrams showing high and low values of the orientation index for portlandite.



Large portlandite crystals
Cement grain
Cement grain
Aggregate or hardened paste

Fig. 14 - Wall effect and crystal growth at aggregate-paste interface.

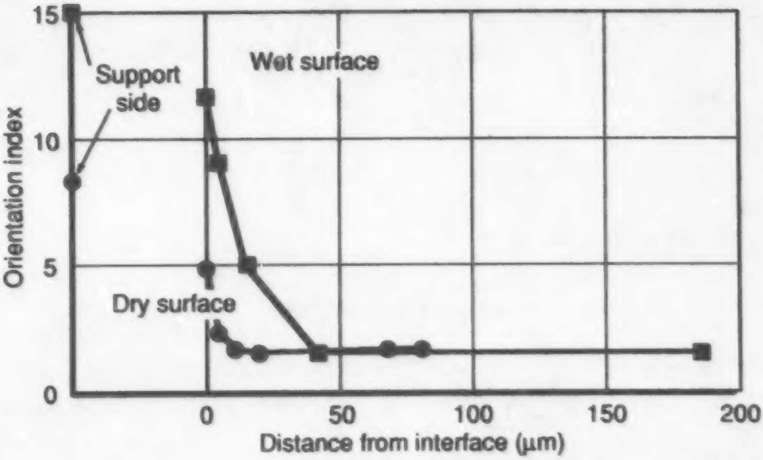


Fig. 15 - Portlandite orientation index profiles (cement A).

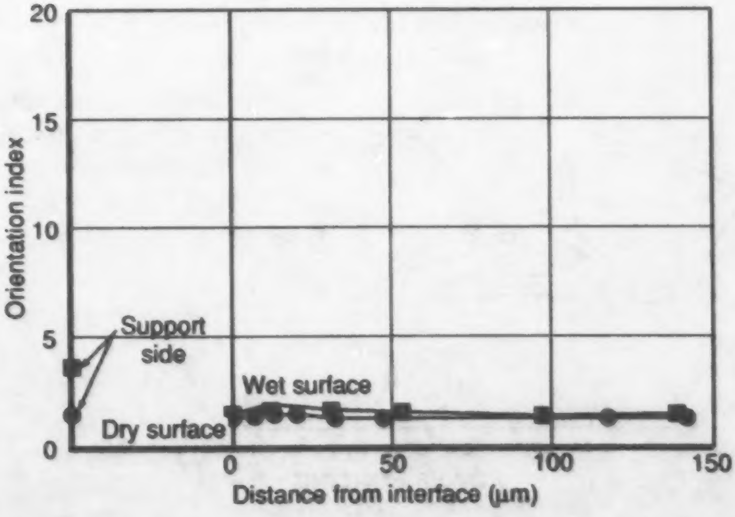
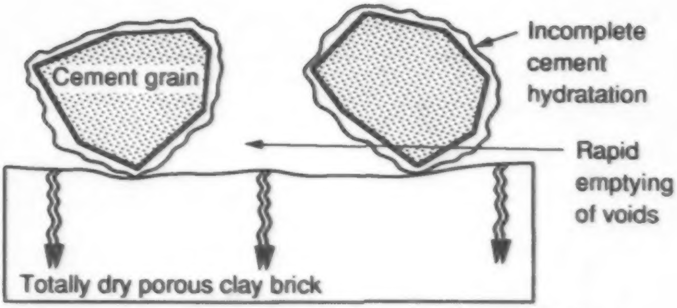
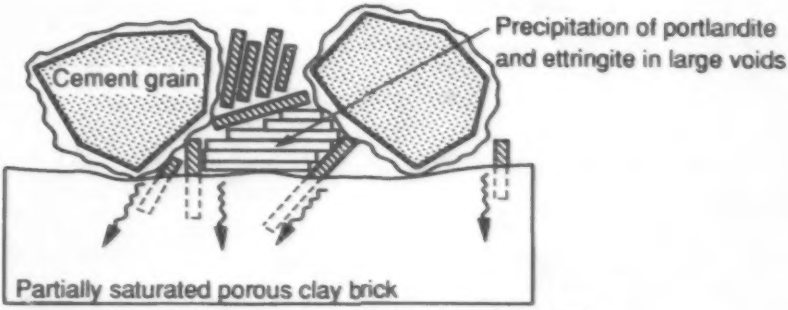


Fig. 16 - Portlandite orientation index profiles (cement B).


Cement grain
Incomplete cement hydratation
Rapid emptying of voids
Totally dry porous clay brick

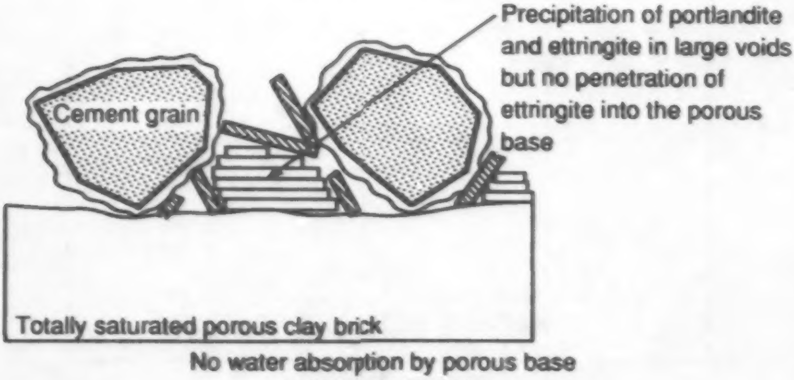
Rapid absorption of water into porosity

ALMOST NO BONDING


Cement grain
Precipitation of portlandite and ettringite in large voids
Partially saturated porous clay brick

Slow absorption of sulfate saturated water
helps crystal growth of ettringite into porosity

GOOD BONDING


Cement grain
Precipitation of portlandite and ettringite in large voids but no penetration of ettringite into the porous base
Totally saturated porous clay brick

No water absorption by porous base

LOWER BONDING

Fig. 17 - Bonding strength on porous base.

(after Grandet, 1971)

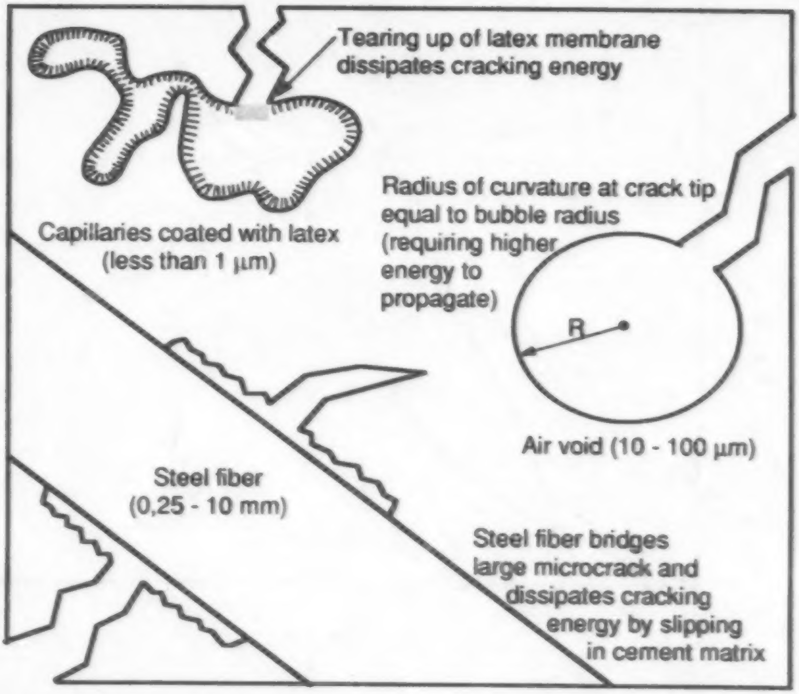

Tearing up of latex membrane
dissipates cracking energy
Radius of curvature at crack tip
equal to bubble radius
(requiring higher
energy to
propagate)
Capillaries coated with latex
(less than 1 μm)
R
Air void (10 - 100 μm)
Steel fiber
(0,25 - 10 mm)
Steel fiber bridges
large microcrack and
dissipates cracking
energy by slipping
in cement matrix

Fig. 18 - Three modes of cracking energy dissipation in cement matrix.

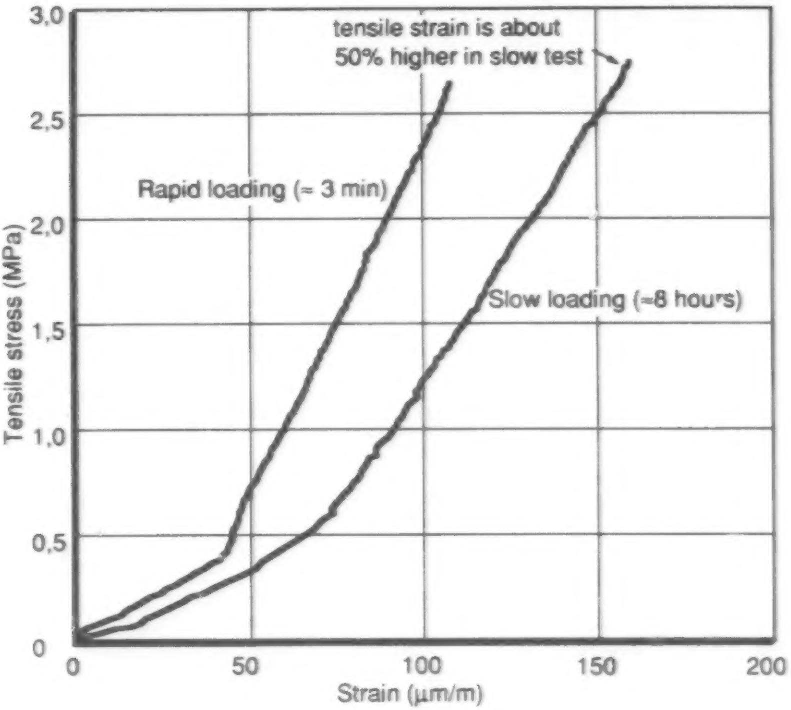


Fig. 19 - Typical stress-strain curves in rapid and slow tensile tests for concrete showing creep in tension.

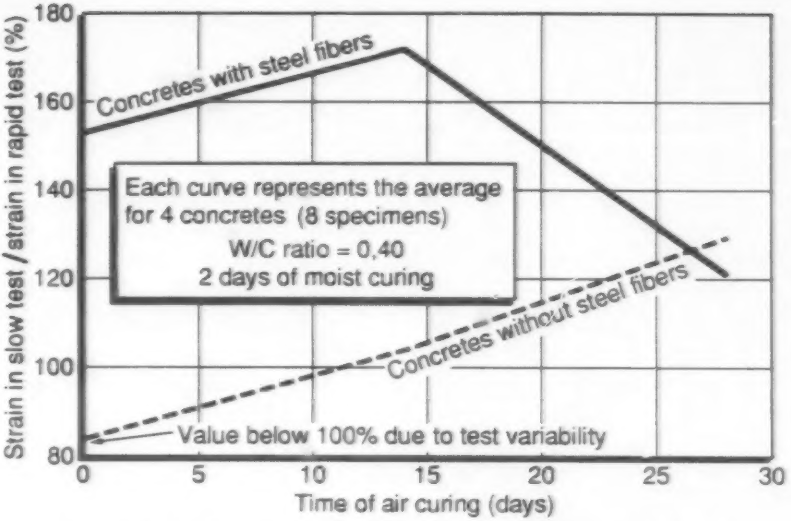


Fig. 20 - Influence of steel fibers on the ratio of the strain in slow tensile tests to the strain in rapid tensile tests.

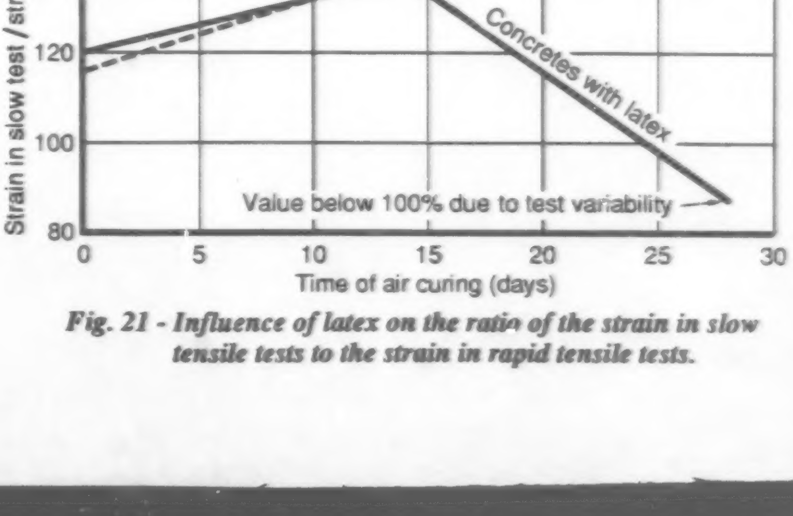


Fig. 21 - Influence of latex on the ratio of the strain in slow tensile tests to the strain in rapid tensile tests.

Recent Research and Development in Concrete-Polymer Composites

By Y. Ohama

Synopsis: Concrete-polymer composites are the relatively new construction materials which are made by improving the properties of conventional cement concrete with polymeric materials. The concrete polymer-composites are usually classified into three types as follows : polymer-modified (or cement) mortar (PCM) and concrete (PCC), polymer mortar (PM) and concrete (PC), and polymer-impregnated mortar (PIM) and concrete (PIC). This paper deals with the recent industry activities, standardization work, research and development in the concrete-polymer composites in the world.

Keywords: cement concrete; polymer cement concrete; polymer concrete; polymer-impregnated concrete; polymers and resins; research and development; specifications; standards; surveys

ACI member Yoshihiko Ohama is a professor of architecture at the College of Engineering , Nihon University, Koriyama, Japan. He has actively been involved in the research and development of concrete-polymer composites for over 30 years. He is vice-president of ICPIC and a member of ACI, ASTM, RILEM, AIJ, JSMS and JTTAS committees.

INTRODUCTION

For the past 40 years, the active research and development of concrete-polymer composites have been conducted all over the world, particularly in the United States, the Soviet Union, the United Kingdom, Japan and Germany. However, it seems that the history of the research and development is considerably different in various countries because of a difference in the background. At present, the concrete-polymer composites are drawing much attention as high-performance or multifunctional materials in the construction industry as well as the mechanical, electrical and chemical industries.

Concrete-polymer composites are the materials which are made by replacing a part or all of the cement hydrate binder of conventional mortar or concrete with polymers, and by strengthening the cement hydrate binder with the polymers. The concrete-polymer composites are generally classified into the following three types by the principles of their process technology :

[1] Polymer-modified (or cement) mortar (PCM) and concrete (PCC)
[2] Polymer mortar (PM) and concrete (PC)
[3] Polymer-impregnated mortar (PIM) and concrete(PIC)

Fig. 1 shows the system and classification of the concrete-polymer composites (1).

Polymer-modified mortar and concrete were developed in the 1920s to 1930s, polymer mortar and concrete in the 1950s, and polymer-impregnated mortar and concrete in the 1960s. Considerable research and development of these three types of concrete-polymer composites were competitively conducted in the 1970s. In the 1980s, the polymer-modified mortar and concrete, and polymer mortar and concrete became the dominant materials. At present, the polymer-modified mortar and concrete, and polymer mortar and concrete are used as popular construction materials. By contrast, the polymer-impregnated mortar and concrete are hardly employed because of a poor balance between their performance and cost due to their high processing cost and awkwardness in applying the process, though they have excellent performance. Recently, a strong interest has been shown toward the applications of the concrete-polymer composites to the repair and restoration of damaged reinforced concrete structures.

The present paper reviews the recent industry activities, standardization work, research and development of concrete-polymer composites in various countries.

INDUSTRY ACTIVITIES AND STANDARDIZATION WORK

In recent years, worldwide interest in concrete-polymer composites has become stronger, and since 1967 many international congresses and symposiums have been held as listed in Table 1. Of such congresses and symposiums, the International Congresses on Polymers in Concrete have been held six times since 1975 : in London, Austin (Texas, U.S.A.), Koriyama (Japan), Darmstadt (West Germany), Brighton (U.K.) and Shanghai (China). In 1981 in Koriyama, the formal international organization, the International Congress on Polymers in Concrete (ICPIC) was founded having the objective of holding periodic congresses and disseminating information on the concrete-polymer composites or polymers in concrete. The proceedings at the above six congresses were published in separate volumes(2-7).

In Europe, the Technical Committee (TC)-105-CPC (Concrete-Polymer Composites) of the International Union of Testing and Research Laboratories for Materials and Structures (RILEM) is active in preparing a state-of-the-art report and terminology of concrete-polymer composites. RILEM TC-113-CPT (Test Methods for Concrete-Polymer Composites) is preparing various test methods for the concrete-polymer composites, mainly polymer-modified mortar and concrete, and polymer mortar and concrete. The Federal Länder Technical Committee, Bridge and Structural Engineering of the Federal Minister for Transport in West Germany issued several technical regulations for polymer-modified mortar and concrete, and polymer mortar and concrete as listed below.

In the United States, Committee 548 Polymers in Concrete of the American Concrete Institute (ACI) has held a symposium every two or three years, publishing the respective volumes of the proceedings (8-13). The committee has also issued a state-of-the-art report(14) and a user's guide (15). In 1986, the Society of the Plastics Industries (SPI) organized the Polymer Concrete Committee. The committee is interested in establishing a liaison with ACI Committee 548 and other organizations. The American Society for Testing and Materials (ASTM), under Committee C 09 Concrete and Concrete Aggregates, has established Subcommittee C 09.03.19 Polymer Modified Concrete, and has already published many standards on polymer mortar as described later.

In Japan, the Synthetic-Resins-for-Concrete (or Polymers-in-Concrete) Committee of the Society of Materials Science, Japan (JSMS) and the Polymers-in-Concrete Committee of the Japan Technology Transfer Association (JTTAS) are currently playing the leading role in the field of the concrete-polymer composites. The Plastics Concrete Committee of the Architectural Institute of Japan (AIJ) which had contributed to the field of the concrete-polymer composites for many years published a user's guide (16) and a state-of-the-art report

(17), and was discharged in 1987. The Japan Concrete Institute (JCI) issued standards for testing methods for polymer-modified mortars (18) in 1987 as stated later. The Society of Materials Science, Japan (JSMS) also published a design recommendation for polyester concrete structures (19) in 1985.

Among the countries using concrete-polymer composites, standardization work on their test methods and quality requirements has been in progress in the United States, Japan and the United Kingdom. Tables 2, 3 and 4 give ASTMs (ASTM Standards), JISs (Japanese Industrial Standards) and BSs (British Standards) on the concrete-polymer composites, which have been published up to the present time. In 1989, DIN (Deutsches Institut für Normung) 51290 (Prüfung von Reaktionsharzbeton im Maschinenbau) was published for polymer concrete for the mechanical industry in West Germany. Table 5 lists the JCI Standards for Test Methods for Polymer-Modified Mortars in Japan. Table 6 indicates the SPI Polymer Concrete Test Methods issued in 1987 in the United States.

Tables 7 and 8 show the guides, specifications and technical regulations for concrete-polymer composites which were issued in Japan, the United States, West Germany, Italy and RILEM.

RESEARCH AND DEVELOPMENT OF POLYMER-MODIFIED MORTAR AND CONCRETE

In Japan, polymer-modified mortar is most widely used as a construction material for finishing and repairing works, but polymer-modified concrete is seldom employed because of a poor cost-performance balance. However, the polymer-modified concrete is widely used for bridge deck overlays and patching work in the United States. In particular, it is estimated that each year over 1.2 million m^2 of bridge decks are overlaid with the polymer-modified concrete (15). In recent years, about 60,300 m^3 of the polymer-modified concrete has been placed each year on both new and existing deteriorated concrete structures in the United States (20).

To produce polymer-modified mortar and concrete, mostly polymers in dispersion (latex or emulsion) form are added to ordinary cement mortar and concrete during mixing. The polymer-modified mortar and concrete have considerable attraction since their process technology is very similar to that of the ordinary cement mortar and concrete. Fig. 2 indicates the classification of polymeric admixtures or cement modifiers for the polymer-modified mortar and concrete. The polymer dispersions widely used are styrene-butadiene rubber (SBR) latex, ethylene-vinyl acetate (EVA) and polyacrylic ester (PAE) emulsions in Japan and Europe, and styrene-butadiene rubber (SBR) latex, polyacrylic ester (PAE) emulsion and epoxy resin (EP) in the United States. The annual consumption of the polymer dispersions in Japan has exceeded 100,000 tons in recent years. In Japan and Europe, the epoxy resin is hardly employed as a cement modifier since it is more expensive than most latex or emulsion polymers. However, there are some interesting developments in epoxy-modified mortar and concrete in Germany. The developments include a one-component epoxy resin system for the cement modifier(21) and a dry mortar using epoxy resin (22). The former does not need to add any hardeners at the construction site. The latter is produced as a dry mixture of an epoxy resin with a hardener, cement and aggregates, and can be applied by adding only water in the field. Powdered emulsions, e.g., ethylene-vinyl acetate (EVA) are often employed to improve the workability and adhesion of fresh cement mortar in Japan and Europe. The film formation characteristics of recent powdered emulsions for the polymer-modified mortar have been improved. In Japan, the powdered emulsions are often used to produce prepackaged-type products such as decorative wall coatings and tile adhesives. Various water-soluble polymers such as methyl cellulose (MC), hydroxyethyl cellulose (HEC), polyvinyl alcohol (PVA) and polyacrylate are used as admixtures for plastering work or underwater concreting work. An interesting current topic relating to the applications of water-soluble polymers is the development of MDF (macro-defect-free) cement, whose technical origin is traced back to a study by J.D.Birchall et al. in 1981 (23). MDF cement is composed of hydraulic cement (e.g., ordinary portland cement or alumina cement), water-soluble polymer (e.g., polyvinyl alcohol or polyacrylamide) and proper admixtures, and processed as a highly filled polymer with the cement, utilizing very low water-cement ratios of 10 to 20 % (24, 25). The material has very high flexural strengths of 10 MPa or more, but a very poor water resistance (26, 27). For the purpose of improving the poor water resistance, some attempts such as latex addition (28), coupling agent addition (29) and polymer impregnation (30) have been examined, but it seems that they have not yet succeeded. Consequently, MDF cement products have been used in very few applications. As an exception, the interior wall panels of MDF cement produced by Ube Industries are commercially available in Japan. The panels have decorative protective coatings to prevent the moisture ingress into them. The polymer modification of DSP (densified system containing homogeneously arranged ultrafine particles) material is also tried to improve its mechanical properties (31). As a recent trend, there have been a few attempts to use polymeric dispersions with chemical admixtures such as superplasticizers (32) and silane(33) at low polymer-cement ratios of 5% or less, and new polymeric chemical admixtures for cement concrete may successfully be developed in the near future. In Japan, an interesting flexible, waterproof material which solidifies in water has recently been developed by Moriyoshi et al.(34), and has a potential as a shock-absorbing, waterproof backfill material for tunnels and dams. The material is composed of an asphalt emulsion, a highly water-absorbing polymer emulsion and cement.

In the production of polymer-modified mortar and concrete, amounts of polymers added are normally 5 to 20% as a polymer-cement ratio, P/C (defined as a weight ratio of polymer solids to cement). The typical mix proportions of the polymer-modified mortar widely used in Japan are as follows; cement : fine aggregate (e.g., river sand) = 1:2 to 1:3 (by weight), polymer-cement ratios 5 to 20% and water-cement ratios 30 to 60% corresponding to its workability. The mix proportions of the polymer-modified concrete for bridge deck applications, suggested in the ACI user's guide (15), are as follows : cement content 414 kg/m^3, sand-aggregate ratio 55-65%, polymer-cement ratio 15% and air content less than 6%. The polymer-modified mortar and concrete are usually moist-cured for a few days, and then air-cured under their service conditions. Most polymer-modified mortar is troweled in the field. Polymer-modified mortar-based decorative wall coatings are generally applied by use of spray guns. The polymer-modified concrete is placed in the same manner as ordinary cement concrete.

For the purpose of improving the workability, drying shrinkage and durability of fiber reinforced cement and concrete, or increasing the flexural strength, toughness and impact resistance of polymer-modified mortar and concrete, fiber reinforced polymer-modified cement and concrete are produced using steel, glass, polymer and carbon fibers. In particular, the polymer modification of glass fiber reinforced cement is very effective in improving its durability (35, 36). Recently, a strong interest has been oriented toward the polymer modification of carbon fiber reinforced cement (37, 38).

Table 9 shows some interesting research and development activities of polymer-modified mortar and concrete in Japan (39).

In Japan, polymer-modified mortar and sometimes paste are popularly used in various applications as shown in Table 10. Because the rapid deterioration of reinforced concrete structures has become a serious problem, a strong interest is focused on the polymer-modified mortar and paste as repair materials, and there is a growing demand for them. This trend is similar to that in other advanced countries. As stated above, about 60,300 m^3 per year of polymer-modified concrete is employed for the repairing work for damaged concrete structures in the United States. In the United Kingdom, about £ 500 million is spent each year on the repair of the concrete structures (40). In connection to such a trend, the carbonation, chloride ion penetration and oxygen diffusion of polymer-modified mortar and concrete have been most actively investigated in various countries (41, 42, 43). According to the typical results from such investigations, generally their porosity or pore volume reduces in the larger radii of 0.2 μm or more, and increases greatly in the smaller radii of 75nm or less as compared to unmodified mortar and concrete because of the effects of filling and sealing their pores with polymers. As a result, their resistance to carbonation, chloride ion penetration or oxygen diffusion is markedly improved. Consequently, the polymer-modified mortar and concrete can be recommended as low-cost, promising materials for preventing chloride-induced corrosion and repairing damaged reinforced concrete structures. In their practical applications, the potential importance of property mismatch between repair materials and the reinforced concrete substrates has been highlighted (40, 44).

About ten years ago, the corrosion of structural steel supports due to chloride ion liberated from the vinylidene chloride-vinyl chloride-modified mortar which had been used to repair an old brick building led to a civil suit (45); therefore, the vinylidene chloride-vinyl chloride emulsion is not currently used as a cement modifier in the United States. In a laboratory study, the liberation of chloride ions is observed in such a chlorine-containing polymer, and a possibility of the corrosion of the reinforcing bars which is induced by the chloride ions in reinforced concrete structures is suggested (46).

Recently, polymer-modified paste or slurry with high polymer-cement ratios of 50% or more has widely been used as a liquid-applied compound for waterproofing membrane coatings for apartment houses, silos, underground tanks, etc., in Japan and Germany (47, 48). The paste or slurry consists of portland cement or alumina cement, sand, polymer dispersion, water and various additives, and forms flexible waterproofing membranes with thicknesses of 2 to 4mm. In Japan, more than 2 million m^2 of reinforced concrete substrates are applied each year by the paste or slurry.

RESEARCH AND DEVELOPMENT OF POLYMER MORTAR AND CONCRETE

Currently, polymer mortar, like polymer-modified mortar, is popularly used for finishing work, and polymer concrete is generally employed as precast products in various countries. In Japan and Europe, the polymer mortar and concrete have already been the dominant construction material in the 1970s ; however, in the United States, they became the dominant construction material in the 1980s.

Commercially available binders for polymer mortar and concrete include various thermosetting resins, tar-modified resins, resin-modified asphalts and vinyl monomers. Fig. 3 shows the classification of polymeric binders or liquid resins for the polymer mortar and concrete. In Japan, binders for the polymer mortar are chiefly epoxy resin (EP), unsaturated polyester resin (i.e., polyester-styrene system)(UP), vinyl ester resin (VE) and methyl methacrylate monomer (MMA), and the most common binder for the polymer concrete is the unsaturated polyester resin. The most common types of the binders in the United States and Western Europe are methyl methacrylate monomer, unsaturated polyester and epoxy resins, and the methyl methacrylate monomer and unsaturated polyester resin have the lowest cost. Furan resin (mainly furfural-acetone resin) is widely used for the same purpose in the Soviet Union and Eastern Europe. The use of methyl methacrylate monomer is limited because of its higher flammability and disagreeable odor. However, it has received much attention on account of the good workability and low-temperature curability of the polymer mortar and concrete made from it. In recent years, some trials on the application of recycled monomers or polymers to the polymeric binders or liquid resins have been done. The typical examples of the recycled monomers or polymers used are methyl methacrylate from acrylic molding products (49) and unsaturated polyester resin from PET (polyethylene terephthalate) beverage bottles (50).

Fillers such as ground calcium carbonate, silica and fly ash are used to extend binders and to fill the very fine particles for aggregates. Coarse and fine aggregates used for polymer mortar and concrete are river gravels and sands, crushed stones such as andesite, limestone (or marble) and granite, silica sands, etc., which should be clean and sound, and have low moisture content. The moisture content is recommended to be 0.5% or less in the AIJ user's guide (16), or less than 1.0% in the ACI user's guide (15).

In the structural applications of polymer mortar and concrete in Japan and the United States, mild steel bars (for conventional cement concrete), PC bars (i.e., high tensile strength steel bars for prestressed concrete) and FRP (fiber reinforced plastics) rods are used to reinforce the members (51, 52), and steel fibers, glass fibers, etc. are done to reinforce the polymer mortar and concrete themselves (53, 54). The use of the polymer mortar as a matrix for ferrocement has recently been tried in Japan (55).

Amounts of polymers as binders in polymer mortar and concrete are normally 9 to 25 wt%. The properties of the polymer mortar and concrete depend largely on the types and properties of the polymer binders and the properties of aggregates. Regardless of the binder types, their typical mix proportions (by weight) widely used in Japan are as follows; binders : fillers : fine aggregates = 1:(0 to 1.5):(3 to 7) for the polymer mortar, and binders : fillers : fine and coarse aggregates mixtures = 1:(1 to 1.5):(8 to 8.5) for the polymer concrete. An interesting mix proportioning system for polymer concrete is reported in Poland(56).

Generally, polymer mortar and concrete are mixed by use of forced mixing-type batch mixers, and cast into forms or molds in a manner similar to that for conventional cement mortar and concrete. The polymer mortar is often troweled as surface coatings or linings. Refined continuous mixers with automatically variable metering of the materials used are also available, and have relatively higher output rates and much lower maintenance requirements than batch mixers. A continuous mixer with an output rate of 250 kg/min has been developed in West Germany, and by using it an overlay placement system has been done in the United States (57). In recent years, a polymer concrete placement equipment with impact girder has been developed for repair work in the United States as shown in Fig. 4(20). The impact girder is attached to provide a moving protection for the workers. In Japan, a small-diameter (inner diameter, 1200 mm) automated shield tunneling system for telecommunication cable lines as illustrated in Fig. 5 was successfully developed using a quick-setting polyester mortar (58). A company in Japan has developed a shotcrete system with epoxy mortar for the repairs of reinforced concrete structures (59). In the cast-in-place applications of the polymer mortar and concrete, prepacked-type (in which a liquid resin as a binder is poured into prepacked aggregates) and prepackaged-type (in which two packages are supplied, and their contents are mixed in the field) products tend to be widely used. In the applications of the polymer mortar and concrete, polymerization and curing processes are usually accomplished through an initiator-promoter system. The polymer mortar and concrete are subjected to ambient temperature cure or heat cure. Silicone greases, oils and emulsions, paraffin wax and polyethylene sheets are frequently used as mold-releasing agents.

Recently, polymer mortar and concrete for underwater construction have been developed in Japan (60). They are produced by using methyl methacrylate monomer as a binder, and can be placed and bonded underwater. As a recent development of the polymer concrete in the United States, electrically conductive polymer concrete overlay which is used with a cathodic protection system for bridge decks has been reported (61).

Recently, research has been directed toward the fracture mechanics of polymer mortar and concrete, and several reports have been published on this subject (62, 63, 64).

Table 11 gives some interesting research and development activities of polymer mortar and concrete in Japan (39).

In Japan, polymer mortar is used as a popular construction material in various applications as listed in Table 12. At present, polyester concrete is most widely used for various structural and nonstructural precast products, and has few cast-in-place applications. Table 13 gives various applications of the polyester concrete. Table 14 shows the production of major precast polymer mortar and concrete products in Japan. In the United States, polymer mortar and concrete are chiefly employed as patching materials for repairing works and overlays for bridge decks in the cast-in-place applications, and in the precast applications as listed in Table 15 (15). In Europe, the precast applications such as machine tool structures, building panels, utility boxes and underground junction boxes of the polymer concrete advance, and especially the development of the machine tool structures is active in Germany and Switzerland.

It is most important that the developments of mass production systems for precast polymer concrete and of automated application systems for cast-in-place polymer concrete should aim at cost reduction, and that a good balance between the economy and performance should be achieved.

RESEARCH AND DEVELOPMENT OF POLYMER-IMPREGNATED MORTAR AND CONCRETE

Currently, polymer-impregnated mortar and concrete are hardly used as construction materials, and their development is not being actively pursued. The reasons for this are considered as follows : [1] Their process technology is complicated, in addition, a great deal of thermal energy is consumed in base mortar and concrete drying and polymerization processes, and consequently their process cost is high; [2] Although their performance is almost similar to that of polymer mortar and concrete, a balance between their cost and performance is poor; and [3] No accurate, easy measuring methods for polymer impregnation depth are known, their quality control is difficult, and therefore their reliability is poor in structural applications. At present, a few manufacturers of the polymer-impregnated mortar and concrete precast products in Japan experimentally produce low-and intermediate-level radioactive waste containers, interlocking blocks, permanent forms, etc. However, the polymer-impregnated mortar and concrete are not now produced in any appreciable quantities in the United States and other countries. The last application of the polymer-impregnated concrete in the United States was the field polymer impregnation of the roadway of the Grand Coulee Dam at a cost of US $ 51/m^2 in 1982 (65).

Table 16 lists some interesting research and development activities of polymer-impregnated concrete in Japan.

CONCLUSIONS

The concrete-polymer composites which have been developed in various countries till now are classified into polymer-modified mortar and concrete, polymer mortar and concrete, and polymer-impregnated mortar and concrete. Currently, the polymer-modified mortar and concrete, and polymer mortar and concrete are used as construction materials, while the polymer-impregnated mortar and concrete are seldom employed because of an undesirable cost-performance balance in their practical applications. A great interest in advanced countries is now oriented toward the use of the polymer-modified mortar and concrete, and polymer mortar and concrete as repair materials for deteriorated reinforced concrete structures, and done toward the precast products using the polymer mortar and concrete. Standardization work on the test methods, quality requirements, guides and specifications of the polymer-modified mortar and concrete, and polymer mortar and concrete has been in progress in the United States, Japan, Germany, the United Kingdom and international organization such as RILEM.

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Table 1 Major International Congresses and Symposiums on Polymers in Concrete.

Year	Venue	Congress or Symposium Name
1967	Paris, France	RILEM Symposium,"Synthetic Resins in Building Construction"
1973	Tyne, U.K.	Symposium,"Resins and Concrete"
1975	London, U.K.	"First International Congress on Polymers in Concrete"
1978	Austin, Texas, U.S.A.	"Second International Congress on Polymers in Concrete"
1981	Koriyama, Japan	"Third International Congress on Polymers in Concrete"
1981	Prague, Czechoslovakia	ICP/RILEM/IBK International Symposium,"Plastics in Material and Structural Engineering"
1984	Liege, Belgium	International Symposium,"Future for Plastics in Building and in Civil Engineering"
1984	Darmstadt, West Germany	"Fourth International Congress on Polymers in Concrete"
1986	Prague, Czechoslovakia	International Symposium,"Mechanics of Polymer Composites"
1986	Aix-en-Provence, France	RILEM International Symposium,"Adhesion between Polymers and Concrete"
1987	Brighton, U.K.	"Fifth International Congress on Polymers in Concrete"
1990	Shanghai, China	"Sixth International Congress on Polymers in Concrete"
1991	Bochum, Germany	"International Symposium on Concrete-Polymer Composites"
1991	San Francisco, California, U.S.A.	"ACI-ICPIC North American Workshop on Polymers in Concrete"

Table 2 ASTM Designations for Polymer Mortars and Materials.

ASTM C 267	Standard test method for chemical resistance of mortars, grouts, and monolithic surfacings
ASTM C 307	Standard test method for tensile strength of chemical-resistant mortars, grouts, and monolithic surfacings
ASTM C 308	Standard test methods for working and setting times of chemical-resistant resin mortars
ASTM C 321	Standard test method for bond strength of chemical-resistant mortars
ASTM C 395	Standard specification for chemical-resistant resin mortars
ASTM C 399	Standard practice for use of chemical-resistant resin mortars
ASTM C 413	Standard test method for absorption of chemical-resistant mortars, grouts, and monolithic surfacings
ASTM C 531	Standard test method for linear shrinkage and coefficient of thermal expansion of chemical-resistant mortars, grouts, and monolithic surfacings
ASTM C 579	Standard test methods for compressive strength of chemical-resistant mortars, grouts, and monolithic surfacings
ASTM C 580	Standard test method for flexural strength and modulus of elasticity of chemical-resistant mortars, grouts, and monolithic surfacings
ASTM C 658	Standard specification for chemical-resistant resin grouts
ASTM C 881	Standard specification for epoxy-resin-base bonding systems for concrete
ASTM C 882	Standard test method for bond strength of epoxy-resin systems used with concrete
ASTM C 884	Standard test method for thermal compatibility between concrete and an epoxy-resin overlay
ASTM C 905	Standard test methods for apparent density of chemical-resistant mortars, grouts, and monolithic surfacings
ASTM D 3262	Standard specification for reinforced plastic mortar sewer pipe
ASTM D 3517	Standard specification for reinforced plastic mortar pressure pipe
ASTM D 3840	Standard specification for reinforced plastic mortar pipe fittings for nonpressure applications

Table 3 JIS for Polymer-Modified Mortars and Polymer Concretes.

JIS A 1171	Method of making test sample of polymer-modified mortar in the laboratory
JIS A 1172	Method of test for strength of polymer-modified mortar
JIS A 1173	Method of test for slump of polymer-modified mortar
JIS A 1174	Method of test for unit weight and air content (gravimetric) of fresh polymer-modified mortar
JIS A 6203	Polymer dispersions for cement modifiers
JIS A 6906	Wall coatings for thin textured finishes
JIS A 6910	Multi-layer wall coatings for glossy textured finishes
JIS A 6915	Wall coatings for thick textured finishes
JIS A 6916	Cement filling compound for surface preparation
JIS A 6917	Lightweight aggregate coating materials
JIS A 1181	Method of making polyester resin concrete specimens
JIS A 1182	Method of test for compressive strength of polyester resin concrete
JIS A 1183	Method of test for compressive strength of polyester resin concrete using portions of beams broken in flexure
JIS A 1184	Method of test for flexural strength of polyester resin concrete
JIS A 1185	Method of test for splitting tensile strength of polyester resin concrete
JIS A 1186	Measuring methods for working life of polyester resin concrete
JIS A 5350	Fiberglass reinforced plastics mortar pipes
JIS K 6931	Reclaimed plastics bars and rods, plates and piles
JIS K 6932	Reclaimed plastics piles for survey and boundary

Table 4 BS for Concrete-Polymer Composites.

BS 6319	Testing of resin compsitions for use in construction
BS 6319; Part 1: 1983	Method for preparation of test specimens
BS 6319; Part 2: 1983	Method for measurement of compressive strength
BS 6319; Part 3: 1983	Method for measurement of flexural strength
BS 6319; Part 4: 1984	Method for measurement of bond strength (slant shear method)
BS 6319; Part 5: 1984	Method for determination of density of hardened resin compositions
BS 6319; Part 6: 1984	Method for determinataion of modulus of elasticity in compression
BS 6319; Part 7: 1985	Method for measurement of tensile strength
BS 6319; Part 8: 1984	Method for the assessment of resistance to liquids

Table 5 JCI Standards for Test Methods for Polymer-Modified Mortars.

(1) Method of test for setting time of polymer-modified mortar
(2) Method of test for tensile strength of polymer-modified mortar
(3) Method of test for shear strength of polymer-modified mortar
(4) Method of test for flexural strength and flexural toughness of polymer-modified mortar
(5) Method of test for adhesion of polymer-modified mortar
(6) Method of test for adhesion durability of polymer-modified mortar after warm-cool cycling
(7) Method of test for impact resistance of polymer-modified mortar
(8) Method of test for abrasion resistance of polymer-modified mortar
(9) Method of test for resistance of polymer-modified mortar to rapid freeezing and thawing
(10) Method of test for incombustibility of polymer-modified mortar
(11) Method of test for reistance of polymer-modified mortar to accelerated carbonation
(12) Method of test for chloride ion penetration depth of polymer-modified mortar
(13) Method of test for compressive strength and modulus of elasticity of polymer-modified mortar
(14) Method of test for thermal expsansion of polymer-modified mortar
(15) Method of test for bond of polymer-modified mortar to reinforcing bar
(16) Method of test for chemical resistance of polymer-modified mortar
(17) Method of test for corrosion-inhibiting property of polymer-modified mortar

Table 6 SPI Polymer Concrete Test Methods.

(1.0)	Test method for making and curing polymer concrete specimens
(2.0)	Test method for compressive strength of cylindrical polymer concrete specimens
(3.0)	Test method for compressive strength of polymer concrete cubes
(4.0)	Test method for splitting tensile strength of polymer concrete specimens
(5.0)	Test method for flexural strength of polymer concrete (using simple beam with third-point loading)
(6.0)	Test method for compressive strength of polymer concrete using portions of beams broken in flexure
(7.0)	Test method for flexural strength of polymer concrete (using simple beam with center-point loading)
(9.0)	Test method for creep of polymer concrete in compression
(10.0)	Test method for specific gravity and water absorption of polymer concrete
(11.0)	Test method for flammability of polymer concrete
(13.0)	Test method for resistance of polymer concrete to chemical reagents
(14.0)	Test method for impact resistance of polymer concrete by means of a TUP (falling weight)

Note : 8.0 and 12.0 have not been published.

Table 7 Guides and Standard Specifications of Concrete-Polymer Composites in Japan.

Institution or Organization	Guide and Standard Specification
Society of Materials Science, Japan (JSMS)	Recommendation for design of polyester resin concrete structures (1985)
Architectural Institute of Japan (AIJ)	Guide for the use of concrete-polymer composites (1987) JASSs (Japanese Architectural Standard Specifications) including the concrete-polymer composites JASS 8 (Water proofing and sealing) (1986) JASS 15 (Plastering work) (1989) JASS 18 (Paint work) (1989) JASS 23 (Spray finishing) (1989)

Table 8 Guides and Standard Specifications of Concrete-Polymer Composites in the United States, West Germany, Italy and RILEM.

Institution or Organization	Guide and Standard Specifications
RILEM Technical Committee 52-RAC Resin Adhesion to Concrete (Int.Org.)	Recommended Tests to Measure the Adhesion Properties between Resin Based Materials and Concrete (1986)
ACI Committee 548 (U.S.A.)	Guide for the Use of Polymers in Concrete (1986)
Forschungsgesellschaft für das Strassenwesen (West Germany)	Merkblatt für die Unterhaltung und Instandsetzung von Fahrbahndecken aus Beton Teil: Ausbesserung von Oberflächen-und Kantenschäden mit Reaktionsharzmörtel (1978) Merkblatt für die Erhaltung von Betonstrassen (1985)
The Federal Minister for Transport, The Federal Länder Technical Committee, Bridge and Structural Engineering (West Germany)	ZTV-SIB 87 Supplementary Technical Regulations and Guidelines for the Protection and Maintenance of Concrete Components (1987) TP BE-PCC Technical Test Regulations for Concrete Replacement Systems Using Cement Mortar/Concrete with Plastics Additive (PCC) (1987) TP BE-PC Technical Test Regulations for Concrete Replacement Systems Using Reactive Resin Mortar/Reactive Resin Concrete (PC) (1987) TL BE-PCC Technical Delivery Conditions for Concrete Replacement Systems Using Cement Mortar/Concrete with Plastics Additive (PCC) (1987) TL BE-PC Technical Delivery Conditions for Concrete Replacement Systems Using Reactive Resin Mortar/Reactive Resin Concrete (PC) (1987)
Deutschen Beton-Vereins E.V. (DBV) (West Germany)	Merkblatt Instandsetzung von Betonbauteilen (1982) Merkblatt Anwendung von Reaktionsharzen im Betonbau (1984)
Associazione Italiana Cemento Armato Precompresso (AICAP) (Italy)	Raccomandazioni tecniche A.I.C.A.P. per l'impiego del conglomerato cementizio impregnato con polimeri (1985)

Table 9 Research and Development Activities of Polymer-Modified Mortar and Concrete in Japan.

Topic	Outline
1. Durability of PCM	(1)Examination on carbonation, chloride ion (Cl^-) or oxgen (O_2) diffusion behavior of PCM (2)Study on fire resistance of PCM (3)Inhibition of alkali-aggregate reaction by PCM
2.Reparing systems using PCM for reinforced concrete structures	(1)Development of repairing systems using PCM and corrosion alkalinity-inhibitors (2)Development of repairing systems using PCM and alkalinity-imparitng agents
3.Polymer modification for fiber reinforced mortar and concrete, and ferrocement	(1)Polymer modification of fiber reinforced mortar and concrete using steel, glass, polymer and carbon fibers (2)Development of PCM permanent forms using PAE emulsion and polyethylene nets (3)Polymer modification of ferrocement
4.Very rapid hardening PCM and PCC	(1)Development of shotcreting system using magnesium acrylate, having a setting time of 1 second or less (2)Development of SBR-modified ultrarapid hardening cement concrete for repair materials and overlays
5.Artificial wood	(1)Development of calcium silicate-SBR latex-glass fibers mixtures (2)Development of portland cement-fly ash-SBR latex-carbon fibers mixtures
6.Underwater concrete	Development of underwater concreting systems using cellulose- or acrylate-based water-soluble polymers
7.High-strength concrete using highly water-absorbing polymers	Development of high-strength concrete using highly water-absorbing polymers such as polyacrylates
8.Polyemric admixtures for highly durable concrete	Developoment of polymeric admixtures consisting of polymer dispersions and chemical admixtures such as alkyl alkoxy silane and $Ca(NO_2)_2$ for highly durable concrete

Table 10 Applications of Polymer-Modified Mortar (or Paste) in Japan.

Application	Location of Work
Floorings and pavements	Floors for houses, warehouses, schools, hospitals, offices, shops, toilets, passages, stairs, gymnasiums and factories, garages, railway platforms, roads, airport runways, etc.
Integral waterproofings	Concrete roof-decks, mortar walls, concrete block walls, water tanks, swimming pools, septic tanks, silos, etc.
Adhesives	Tile adhesives, adhesives for floorings, walling materials and heat-insulating materials, adhesives for joining new cement concrete or mortar to old cement concrete or mortar, etc.
Decorative coatings (including surface-preparing materials)	Wall coatings, lightweight aggregate coating materials, cement filling compounds and self-levelling cements for surface preparation, etc.
Repair materials	Grouts for repairing cracks and delaminations of concrete structures, patching materials for damaged concrete structures, protective coatings for corroded reinforcing bars, etc.
Anti-corrosive linings	Effluent drains, chemical or machinery plant floors, grouts for acid-proof tiles, floors for chemical laboratories and pharmaceuticals warehouses, septic tanks, hot spring baths, etc.
Deck coverings	Internal and external ship-decks, bridge decks, footbridge decks, train floors, etc.

Table 11 Research and Development Activities of Polymer Mortar and Concrete in Japan.

Topic	Outline
1. Mix design systems of PM and PC	(1) Mix design systems for polyester mortar and concrete (2) Mix design systems for polymethyl methacrylate mortar and concrete (3) Development of ready-mixed polyester concrete
2. Fiber reinforcement of PM and PC	(1) Steel fiber reinforcement (2) Glass cloth reinforcement
3. Use of wet aggregates for PC	Use of wet aggregates having a moisture content of up to 3.0% with silane coupling agent, moisture absorbent and steel fibers, providing improvements in strength and freeze-thaw durability
4. Low-temperature curability of PC	Low-temperature cure of polymethyl methacrylate concrete at 0 to -20°C
5. Long-term durability of PM and PC	Long-term durability of polyester mortar and concrete under water
6. Automated application systems of PM and PC	(1) Small-diameter shield tunneling system using quick-setting polyester mortar or hard-foamed polyurethane (2) Shortcreting system using epoxy mortar
7. Artificial marble tiles and panels	Development of artificial tiles and panels made with polymer pastes with flame-retarding fillers such as aluminium hydroxide and magnesium hydroxide
8. Applications of PM and PC to machine tool structures	Examination on dynamic performance and thermal deformation, and trial production of machine tools

Table 12 Applications of Polymer Mortar in Japan.

Application	Location of Work
Floorings (including decorative finishings)	Floors for houses, warehouses, offices, schools, hospitals, factories, shops, toilets, passages and stairs, garages, railway platforms, train floors, etc.
Pavements	Roads (footways and traveled ways), bridge decks, footbridge decks, parking lots, airport runways, etc.
Anti-corrosive linings	Effluent drains, chemical or machinery plant floors, grouts for acid-proof tiles, floors for chemical laboratories and pharmaceutical warehouses, septic tanks, electrolytic baths, hot spring baths, offshore structures, e.g., piers and sea berths, etc.
Adhesives	Adhesives for floorings, walling materials and heat-insulating materials, tile adhesives, adhesives for joining new cement concrete or mortar to old cement concrete or mortar, embedment of anchor bolts, etc.
Repair materials	Grouts for repairing cracks and delaminations of concrete structures, patching materials for damaged concrete structures, protective coatings for corroded reinforcing bars, etc.
Integral waterproofings	Concrete roof-decks, mortar walls, concrete blocks, water tanks, swimming pools, septic tanks, silos, etc.
Small-diameter automated shield tunneling system	Telecommunication cable lines, etc.
Precast products	FRP-reinforced polymer mortar pipes for sewage and irrigation systems, FRP-reinforced polymer mortar panels, vibration isolators, artificial marble tiles and panels, etc.

Table 13 Applications of Polyester Concrete in Japan.

Application	Location of Work
Structural precast products	Manholes and handholes for telecommunication cable lines electric power cable lines and gas pipelines, prefabriacted cellars, tunnel liner segments for telecommunication cable lines and sewerage systems, piles for port or hot spring construction, machine tool structures, e.g., beds and saddles, FRP-reinforced frames or panels for buildings, etc.
Non-structural precast products	Gutter covers, U-shaped gutters, footpath panels, terrazzo tiles and panels and large-sized or curved decorative panels for buildings, partition wall panels, sinks, counters, washstands, bathtubs, speakers for stereophonographs, etc.
Cast-in-place applications	Spillway coverings in dams, protective linings of stilling basins in hydroelectric power stations, coverings of checkdams, foundations of buildings in hot spring areas, etc.

Table 14 Production of Major Precast Polymer Mortar and Concrete Products in Japan.

Type of Precast Product	Type of Binder	Number of Manufacturer	Annual Productions (t)
Manhole for cable lines and pipelines	UP	4	20,000-30,000
FRP-reinforced pipe	UP	4	40,000-50,000
Artificial marble	UP and MMA	Ca.25	30,000-35,000

Table 15 Precast Applications of Polymer Concrete in the United States.

a. Structural and building panels
b. Sewer pipes, equipment vaults, drainage channels, etc.
c. Corrosion resistant tiles, brick, and linings
d. Small water-flow control structures
e. Stair treads and nosings
f. Nonconductive, nonmagnetic support structures for electrical equipment
g. Manhole structures and shims
h. Components for the animal feeding industry
i. Large-scale preinsulated wall panels for segmental building construction
j. Electrical insulators

Table 16 Research and Development Activities of Polymer-Impregnated Mortar and Concrete in Japan.

Topic	Outline
1. Inorganic polymer impregnation	(1)Development of field alkyl alkoxy silane impregnation techniques for the purpose of preventing chloride-induced corrosion for reinforced concrete structures (2)Development of precast inorganic polymer-impregnated mortars using silicon alkoxyds and alkyl alkoxy silanes
2. Rapid field polymer impregnation system	Development of labor- and energy-saving polymethyl methacrylate impreganation system, uisng heat remaining in dried base concrete (process time, 30 to 40 min, polymer impregantion depth, ca.25mm)
3. Superhigh-strength concrete	Development of superhigh-strength concrete made by polymethyl methacrylate impreganation of autoclaved silica fume concrete, having a compressive strength of 250 MPa or more

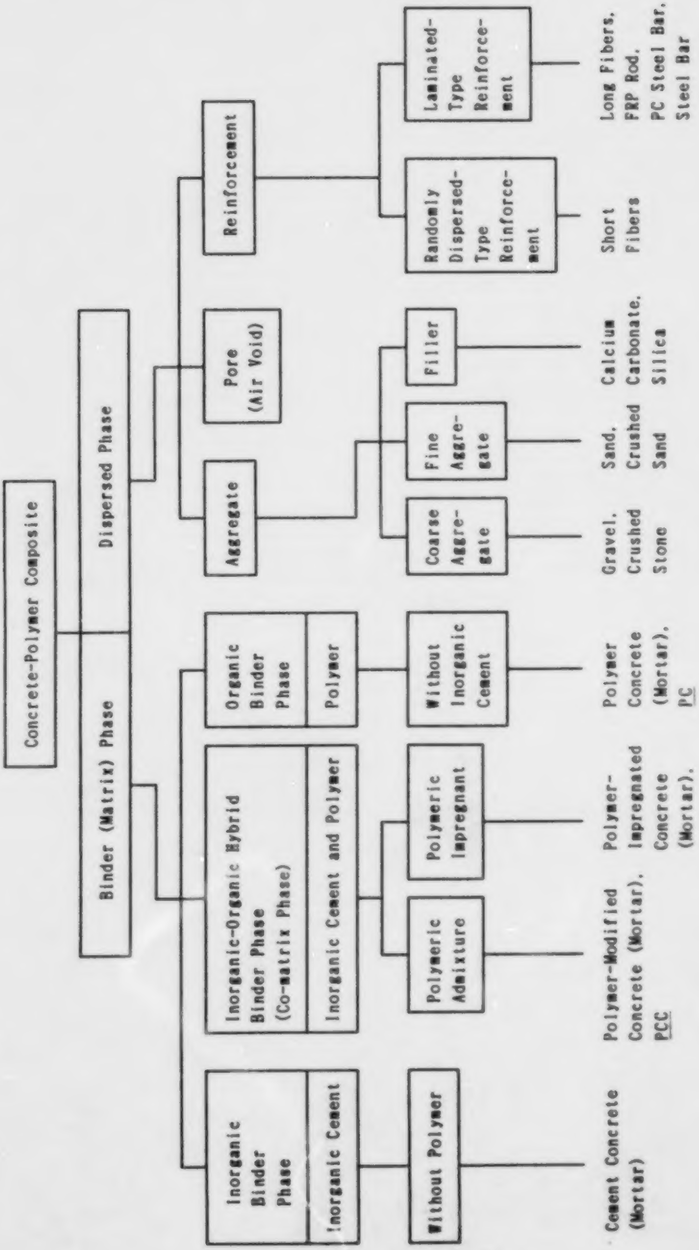
Concrete-Polymer Composite
Binder (Matrix) Phase
Dispersed Phase
Inorganic Binder Phase
Inorganic Cement
Inorganic-Organic Hybrid Binder Phase (Co-matrix Phase)
Inorganic Cement and Polymer
Organic Binder Phase
Polymer
Aggregate
Pore (Air Void)
Reinforcement
Without Polymer
Polymeric Admixture
Polymeric Impregnant
Without Inorganic Cement
Coarse Aggregate
Fine Aggregate
Filler
Randomly Dispersed-Type Reinforcement
Laminated-Type Reinforcement
Cement Concrete (Mortar)
Polymer-Modified Concrete (Mortar). PCC
Polymer-Impregnated Concrete (Mortar). PIC
Polymer Concrete (Mortar). PC
Gravel. Crushed Stone
Sand. Crushed Sand
Calcium Carbonate. Silica
Short Fibers
Long Fibers. FRP Rod. PC Steel Bar. Steel Bar

Fig. 1 System and Classification of Concrete-Polymer Composites.

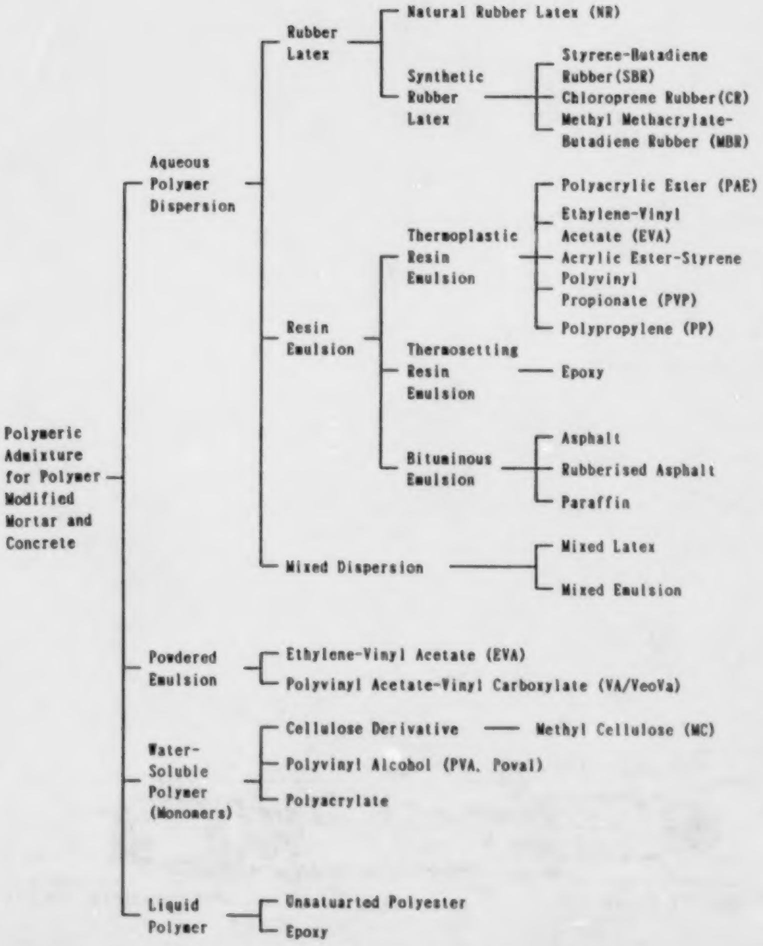
Polymeric Admixture for Polymer Modified Mortar and Concrete
Aqueous Polymer Dispersion
Rubber Latex
Natural Rubber Latex (NR)
Synthetic Rubber Latex
Styrene-Butadiene Rubber(SBR)
Chloroprene Rubber(CR)
Methyl Methacrylate-Butadiene Rubber (MBR)
Resin Emulsion
Thermoplastic Resin Emulsion
Polyacrylic Ester (PAE)
Ethylene-Vinyl Acetate (EVA)
Acrylic Ester-Styrene
Polyvinyl Propionate (PVP)
Polypropylene (PP)
Thermosetting Resin Emulsion
Epoxy
Bituminous Emulsion
Asphalt
Rubberised Asphalt
Paraffin
Mixed Dispersion
Mixed Latex
Mixed Emulsion
Powdered Emulsion
Ethylene-Vinyl Acetate (EVA)
Polyvinyl Acetate-Vinyl Carboxylate (VA/VeoVa)
Water-Soluble Polymer (Monomers)
Cellulose Derivative
Methyl Cellulose (MC)
Polyvinyl Alcohol (PVA, Poval)
Polyacrylate
Liquid Polymer
Unsatuarted Polyester
Epoxy

Fig. 2 Classification of Polymeric Admixtures or Cement Modifiers for Polymer-Modified Mortar and Concrete.

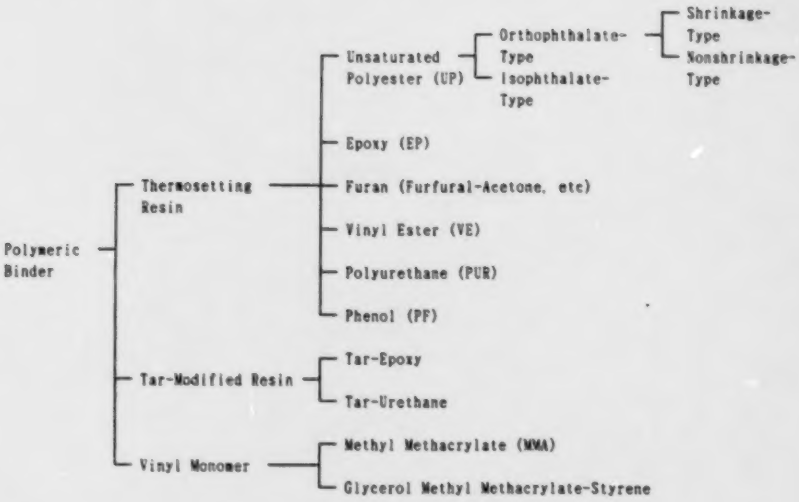
Polymeric Binder
Thermosetting Resin
Unsaturated Polyester (UP)
Orthophthalate-Type
Shrinkage-Type
Nonshrinkage-Type
Isophthalate-Type
Epoxy (EP)
Furan (Furfural-Acetone, etc)
Vinyl Ester (VE)
Polyurethane (PUR)
Phenol (PF)
Tar-Modified Resin
Tar-Epoxy
Tar-Urethane
Vinyl Monomer
Methyl Methacrylate (MMA)
Glycerol Methyl Methacrylate-Styrene

Fig. 3 Classification of Polymeric Binders or Liquid Resins for Polymer Mortar and Concrete.

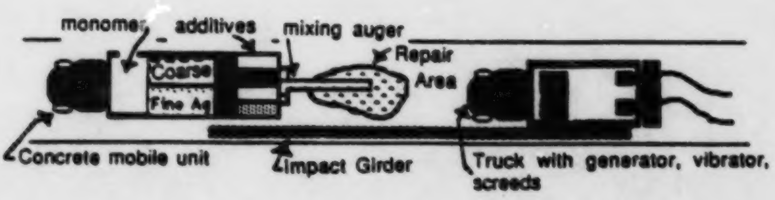
monomer
additives
mixing auger
Coarse
Fine Ag
Repair Area
Concrete mobile unit
Impact Girder
Truck with generator, vibrator, screeds

Fig. 4 Schematic of Polymer Concrete Placement Equipment with Impact Girder.

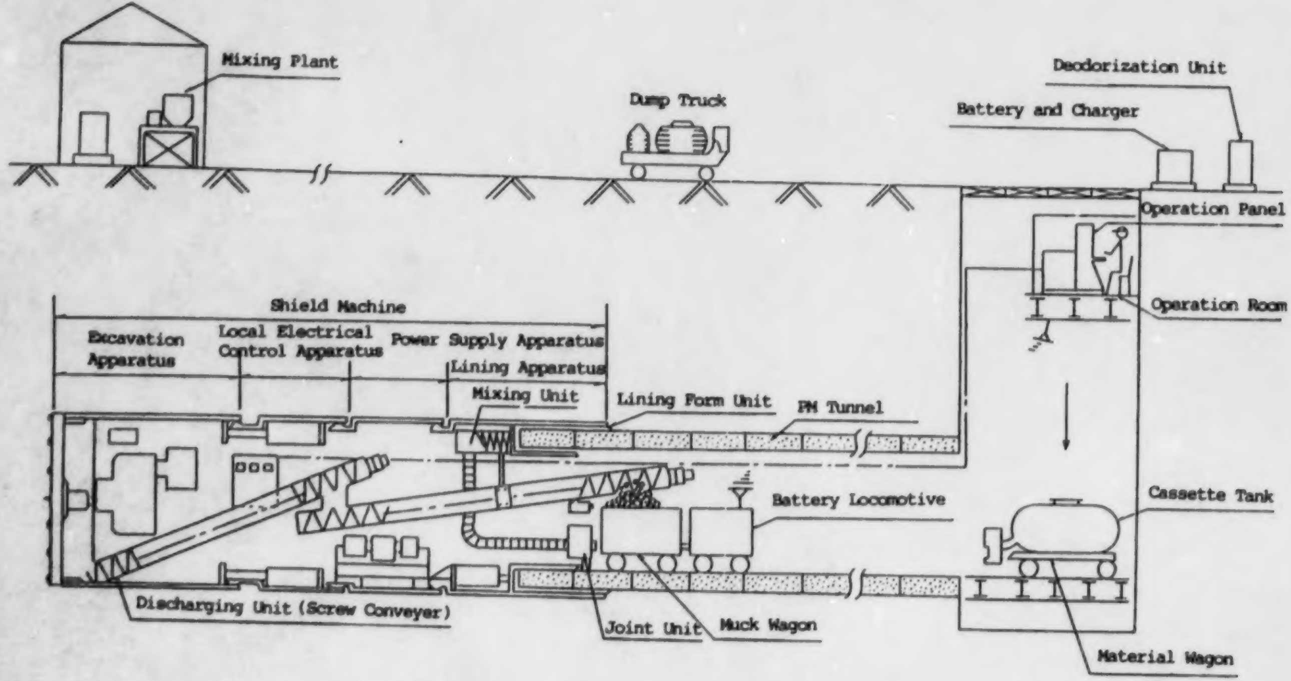
Mixing Plant
Dump Truck
Deodorization Unit
Battery and Charger
Operation Panel
Operation Room
Shield Machine
Excavation Apparatus
Local Electrical Control Apparatus
Power Supply Apparatus
Lining Apparatus
Mixing Unit
Lining Form Unit
PM Tunnel
Battery Locomotive
Cassette Tank
Discharging Unit (Screw Conveyer)
Joint Unit
Muck Wagon
Material Wagon

Fig. 5 Outline of Automated Shield Tunneling System for Telecommunication Cable Lines.

Properties and Applications of Latex-Modified Concretes

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Synopsis: Latex modified concretes (LMC) were developed in order to minimize or to completely eliminate the weakness of portland cement concretes, such as low tensile strength, low ductility, volume instability, and greater than desired permeability. The portland cement concretes are modified with polymer admixtures, such as latexes, powdered emulsions, water soluble polymers, liquid resins, and monomers. These modified concretes may have high strengths, improved bonding to concrete materials, increased flexibility and higher impact resistance, increased resistance to penetration by water and chemicals, improved resistance to frost action, and decreased permeability. Because of the improved performance, these latex-modified concretes are being increasingly used in bridge deck overlays, in the repair and rehabilitation works, and in the construction of new pavements, garages, marine facilities, and tunnel and pipe linings. LMC is more widely used than all other polymer concrete systems due to its less complicated method of making and lower cost than other polymer concrete systems. Basic information about their characterization, past applications and their performances, and prospective new applications are presented.

Keywords: Abrasion resistance; admixtures; air entrainment; bleeding; carbonation pore structure; chemical resistance; compressive strength; creep; ductility; impact bond; impact resistance; latex; liquid resins; permeability; setting time; shrinkage; tensile strength; flexural strength; thermal performance; water retention; workability.

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INTRODUCTION

POLYMERS IN CONCRETE

Portland cement concrete is the most popular and widely used construction material in the world in spite of its low tensile strength, low ductility, volume instability, low strength to weight ratio, and greater than desired permeability. Ever since the introduction of portland cement concrete, attempts are being made to minimize or to completely eliminate these weaknesses. One such attempt is the development of latex modified concrete (LMC), more commonly known as polymer modified concrete (PMC). It is also known as polymer portland cement concrete (PPCC).

The American Concrete Institute (ACI) Committee 548 has defined LMC as portland cement and aggregate combined at the time of mixing with organic polymers that are dispersed or redispersed in water. This organic polymer dispersion in water is called a latex. Depending on the type of latex used, these modified concretes could have increased workability and strengths, improved bonding to concrete substrates, increased flexibility and impact resistance, increased resistance to penetration by water and chemicals, improved resistance to frost action, and decreased permeability (1, 2).

Polymer modified portland cement concretes are more widely used in construction all over the world due to their less complicated method of production and lower cost compared to the other polymer concrete systems like polymer impregnated concrete (PIC) and polymer concrete (PC).

HISTORICAL BACKGROUND

The first reference to latex was in the 16th century, when the Spanish explorers reported that the South American Indians were using rubber latex obtained from the tree Hevea brasilienis to make footwear; but the first patent was obtained in Britain by Cresson on Jan. 12, 1923 (3). The historically important patent was issued for using cement as a filler to paving materials with natural rubber latexes. Geist et al. (4) had conducted a basic study on polyvinyl acetate modified mortar and laid the foundation for valuable research on latex-modified mortar and concrete. Until the early 20th century, the only available latex was natural rubber latex. After the second World War there has been a tremendous increase in the availability of other types of polymers in the market and there have been extensive research and development in latex-modified concrete systems.

The initial investigation on the styrene-butadiene latex-modified portland cement was conducted in 1956 by Dow Chemical Company in Michigan. A co-operative effort between Dow Chemical Company and the Michigan Highway Department resulted in field trials in 1958 (5).

Since the 1960s styrene-butadiene rubber (6), and polyacrylic ester modified concretes have been increasingly used in practical applications. Research and development activities in polymer-modified concrete have been conducted in various countries, including the U.S.A., the U.S.S.R., West Germany, Japan and the U.K. In Japan several standards of the quality and testing methods of latex-type cement modifiers and latex-modified mortars have been issued as Japanese Industrial Standards (2).

LATEXES AND THEIR PROPERTIES

CEMENT MODIFIERS

In recent years various polymer dispersions have been developed and commercialized as cement modifiers. The polymers and monomers commonly used as cement modifiers are shown in Figure 1. Using these cement modifiers, several types of polymer modified concretes like latex, powdered emulsion, water-soluble polymer, liquid resin, and monomer-modified concretes are produced. The most widely used among them is the PMC (7).

WHAT IS A LATEX?

A latex is a stable dispersion of organic polymer particles in an aqueous surfactant solution giving a milky fluid that is generally white to off-white in color. On drying, these particles coalesce to form a continuous film.

An organic polymer is a substance composed of giant molecules, which are formed by the union of a number of simple molecules, known as monomers (7). Polymers can be divided into two groups. One is homopolymer and the other is copolymer. When the polymer is made by the polymerization of one type of monomer, it is called homopolymer, and if it is made by the polymerization of different monomers, it is called copolymer (8).

Natural rubber latex (NR) is a dispersion of polyisoprene, a homopolymer, that is polymerized by the tree.

LATEX TYPES

Latexes are divided into two types by ASTM C1042-85.

Type I: This is a re-emulsifiable latex. This type should be used only in applications which are not subjected to immersion in water or high humidity.
Type II: This is a non re-emulsifiable latex. This type can be used where water immersion or high humidity is expected.

COMMONLY USED LATEXES

Many types and formulations of latexes are manufactured; however, only those specifically developed for use in hydraulic cement are used in concrete applications.

Various latexes that have been used before and those that are currently used for modification of hydraulic cement mixtures are listed below with their commonly used abbreviations.

Elastomeric:	. Natural Rubber (NR)
	. Styrene - Butadiene (SB) (Styrene-Butadiene Rubber-SBR)
	. Polychloroprene (CR) (Neoprene)
	. Acrylonitrile - Butadiene (NBR)
Thermoplastic:	. Polyacrylic Ester (PAE)
	. Styrene - Acrylic (SA)

- * . Ethylene Vinyl acetate (EVA)
- . Vinyl acetate - ethylene (VAE)
- . Polyvinyl Acetate (PVAC)
- . Polyvinylidene Chloride (PVDC)
- . Vinyl acetate-acrylic copolymer(VAC)
- . Polyvinyl Propionate
- . Polypropylene
- . Pure acrylics (e.g. ethyl acrylate/methyl methacrylate)

*Generally in the U.S.A., the name Vinyl acetate - ethylene copolymer (VAE) is used. The same latex is referred to as ethylene vinyl acetate (EVA) in Japan.

Polyvinylidene chloride (PVDC) had been used earlier in the U.S.A and Japan, primarily as a mortar additive and as a concrete admixture. Currently PVDC is not generally used in the U.S.A and Japan because, there is a possibility of corrosion of embedded steel in PVDC mortars and concretes.

Polyvinyl acetates (PVAC) are not generally recommended for use in wet environments and outer exposure because some types may hydrolyze or break down chemically.

MIXED LATEXES

Occasionally blends of different types of latex, like an elastomeric latex with a thermoplastic one, are used. These are known as mixed latexes.

ADVANTAGES OF LATEXES

Latexes are becoming more important because of their health and safety advantage over solvent based products. Some advantages of latexes over solvent based products are

1) nontoxicity
2) non flammability
3) Latexes could be produced at 60 percent solids and still have reasonable viscosity whereas the solids limit on a solvent system is about 10 percent, and
4) Emulsion polymerization permits customization of properties (9).

MATERIALS AND MIXTURE PROPORTIONS

The materials used and the mixing of latex-modified concretes are almost the same as those for conventional portland cement concretes. Latexes are in a dispersed form and they are used in larger quantities in comparison with other chemical admixtures like air-entraining agents, water-reducing agents or superplasticizers.

COMMERCIALLY USED LATEXES

In Japan, the quality of the latex-type cement modifiers is controlled by the Japanese Industrial Standard JISA6203. Table 1 shows the quality requirements for cement modifiers. In the U.S.A, the most commonly used latex for concrete overlays on bridge decks is styrene-butadiene and it has been evaluated for such overlays by the Federal Highway Administration (10). Other latexes in concrete are used for repair and rehabilitation work. The choice of a particular latex may depend on the specific application.

CEMENTS

Portland cements, ASTM Type I and Type II, are commonly used. However, when early strength is required, Type III may be used. When colored concretes are required, white cement is recommended in conjunction with the acrylic or vinyl-acetate latexes. According to their applications, ultra-high-early-strength portland cement, sulfate resisting portland cement, moderate heat portland cement, blended cement and high alumina cements are also used. However, air-entraining cements should not be used.

ADMIXTURES

Air-entraining admixtures should not be used because of the inherent air entrainment due to the latex addition. Water reducers are usually not needed for the latex-modified concretes.

AGGREGATES

The aggregates used for latex modified concretes are similar to those used for quality concrete mixtures. Fine and coarse aggregates such as river sands, gravels, crushed sands and stones, silica sands and artificial lightweight aggregates are used for latex modified concretes. Silica sands and siliceous crushed stones are sometimes recommended for corrosion resistant structures. The aggregates should be clean and sound and properly graded. In some situations, such as when exposed aggregate surfaces are needed, gap graded aggregates could be more efficiently used. The selection of the aggregate size and type is normally dependent on the thickness of the application, type and density of reinforcement as in conventional mixtures.

MIXTURE PROPORTIONING-GENERAL

The mixture proportioning of latex-modified concrete is done in a way similar to that of portland cement concrete because the same variables influence the physical properties of both unmodified and modified concretes. The effect of type and quantity of fine aggregate, coarse aggregate and cement, as well as the quantity of water used, are all similar in both portland cement and latex-modified concretes. However, as a general rule, least amount of water should be used to achieve a suitable, workable consistency as required by the end use. The inclusion of the latex in portland cement replaces some of the water necessary to achieve the workability because latex acts as a dispersant of the portland cement and thus increases flow. In proportioning the mixture, the water in the latex should be taken into consideration. Usually there will be about a 50% water reduction if latex is used.

The mixture proportioning for latex-modified concrete should recognize its improved properties such as tensile and flexural strengths, extensibility, adhesion, durability and watertightness over conventional concrete. These properties are controlled by the polymer-cement ratio rather than the water-cement ratio. Therefore, based on the end use, the polymer-cement ratio is determined.

CONCRETE PROPORTIONS

In latex-modified concrete mixtures, there are usually more fine aggregates than coarse aggregates because overlays, the typical end use, require texturizing of the overlays for skid resistance. The overlays need to be placed on scarified surfaces and the sand helps to prevent segregation at the higher slopes. Higher sand content also reduces bleeding. Low water-cement ratios of less than 0.40, with slumps of 4 to 7 inches (102 to 178 mm), are typical for SBR latex-modified concretes. The maximum air entrainment is limited to 6.5 percent. A typical mixture proportion using SBR latex recommended by the ACI Committee 548A (11) is given in Table 2. Recently by using type III cement and increasing the cement content, new mixture proportions have been developed to produce high early strength latex-modified concrete to speed up overlay construction (12,13). These overlays have been subjected to traffic in less than a day. Table 3 gives the mixture proportions for high early strength latex-modified concrete used by Sprinkel (12) for installation of an LMC overlay on a route 340 bridge in Virginia. A comparison of the chosen proportions with typical proportions used for standard latex-modified concrete and typical unmodified bridge deck concrete is also presented in Table 3.

PROPERTIES OF LATEX-MODIFIED CONCRETES

In general, the properties of fresh and hardened latex-modified concretes show a satisfactory improvement over those of unmodified concretes as discussed below. However these improvements are influenced by various factors that tend to interact with each other. The main factors are: type and quantity of materials used such as latexes, cements and aggregates, polymer-cement ratio, water-cement ratio, binder-void ratio, air content, placement and curing procedures, testing methods, and environmental conditions during the placement and in service.

FRESH CONCRETES

AIR ENTRAINMENT

The surfactants included as emulsifiers and stabilizers in polymer latexes cause an excessive air entrainment in latex-modified concretes compared to unmodified concretes. Air entrainment causes a reduction in strength (approximately a 5 percent reduction in compressive strength for every 1 percent increase in the air content). Therefore the excessive air entrainment is controlled by the addition of an antifoaming agent like silicone. The antifoaming agent is usually added by the latex manufacturer. In addition, the composition of the cement and the type of aggregates can effect the air content. It is difficult to entrain more air when larger size aggregates are used. Therefore it is important to evaluate the actual mixture proportions before using them in the field. The air content of most latex-modified concretes is less than 2 percent (7). The relationship between the polymer-cement ratio and the air content for latex (SBR) modified concrete is shown in Figure 2 (14). For an increase in polymer-cement ratio, there is a variation of the air content in the latex-modified concrete. The addition of air-entraining agent is not required for resistance to freezing and thawing since the latex itself apparently provides this protection. Therefore it is recommended that a specification should include a maximum air content of 6.5 percent, and not a minimum (11).

WORKABILITY

The addition of latex to the portland cement concrete increases the workability. This is caused by the ball bearing action of polymer particles and entrained air and the dispersing effect of the surfactants in the latexes (7). A significant reduction of water-cement ratio can be achieved by the inclusion of latex in concrete as shown in Figure 3 (7). This reduction in water-cement ratio for a constant workability will contribute to increased strength and reduced shrinkage in the hardened concrete. The experimental results reported by Ohama and Kan (15) had shown that for the same workability a considerably reduced water-cement ratio can be achieved with the addition of latex (Table 4).

As in the case of conventional concretes, there is a loss of slump with time in latex-modified concretes. Based on a study by the FHWA (16), it was reported that the slump loss was similar for both conventional concretes and latex-modified concretes.

In a recent study, Kuhlman and Foor have demonstrated that with the addition of SBR latex (latex solids/cement ratio of 0.15) workable concretes with low water cement ratios can be produced using Michigan and Maryland aggregates (17). Both mixtures had fine to coarse aggregate ratio of 1.20/1.04, and a cement factor of 658 lbs. per cubic yard (390 Kg/m^3). A slump of 8 inches (203 mm) was obtained with a water-cement ratio of 0.33 for the Michigan aggregate and a slump of 6 inch (152 mm) was achieved with a water-cement ratio of 0.37 for the Maryland aggregate.

SETTING TIME

It takes slightly more time for the latex-modified concretes to set compared to conventional portland cement concretes. The time difference is dependent on the latex type and quantity used. The setting behavior of SBR latex modified concrete for various latex contents is shown in Figure 4 (14). Similar results were also reported by Smutzer and Hockett (18). The setting time increases with an increase in the latex-cement ratio. Natural rubber latex causes the maximum delay in setting time. The delay in setting time is due, in part, to the fact that surfactants contained in latexes inhibit the hydration of cement (19). Studies by Zivica (20) had shown that in the case of PVAC-modified concrete, the hydration of cement is inhibited by the absorption of the surfactants on the binder surface. Setting time is a function of the hydration of cement and hence a delay in the hydration process is reflected in increased setting time.

WORKING TIME

Working time, which is different from the setting time, is the time required to place, work and finish the surface. It is dependent on drying of the surface. During the drying process, a "crust" will form and it will hinder the proper finishing of the surface. The air temperature, humidity, and wind speed will influence the "crust" formation and will determine working time. The working time is about 15-30 minutes after mixing the latex in the concrete and exposing it to air. The maximum mixing time recommended is 5 minutes and hence the use of transit mixer is not feasible (11).

BLEEDING AND SEGREGATION

There is considerably less bleeding and segregation in latex-modified concretes even when they have high workabilities (flowability characteristics). This is due to the hydrophobic colloidal properties of latexes. The air-entraining and water-reducing effects of surfactants will also contribute to the reduction in bleeding and segregation, resulting in an increase in strength and water-tightness in the hardened products.

FINISHABILITY

The finishing characteristics of latex-modified concretes are different from those of unmodified concretes; they have increased drag and stickiness on the finishing tools. A thin crust of approximately 0.07 inch (2 mm) forms if the surface is allowed to dry. The formation of this crust depends on environmental conditions such as temperature, wind speed and humidity. Therefore it becomes difficult to finish when there is a delay in the finishing operation. After mixing and exposure to air, the finishing time available is generally 15 to 30 minutes (11).

WATER RETENTION

Latex-modified concretes have an excellent water retention property due to hydrophobic and colloidal properties of the latexes and reduced water evaporation. The latex particles fill and seal the voids and form an impermeable latex film and thus retain a sufficient amount of water required for cement hydration. Therefore a dry cure is preferred rather than a wet cure or water cure for latex-modified systems. The water-retention properties also help in preventing the "dry-out" phenomena (the lack of cement hydration due to water loss in the concrete) in thin latex linings and coatings on highly water-absorbable substrates such as dried cement ceramic tiles (7).

PROPERTIES OF HARDENED CONCRETES

EFFECTS OF CURING CONDITIONS ON STRENGTH

The type and duration of curing have significant influence on the strength and rate of development of strength. To obtain maximum strength properties, it is recommended that latex-modified concretes should be cured in 100 percent relative humidity for the first 24 to 48 hours and then dry cured at ambient temperature and humidity. This is unlike unmodified concretes where higher strength properties are achieved by wet-curing. The air curing allows any excess water to evaporate and allows the formation of the latex film which is desirable because the formation of the latex film in the internal structure is the main reason for the improved properties in the latex-modified concretes.

The effects of curing conditions on the compressive and flexural strengths of latex-modified concretes are reported by Ohama (21) as shown in Figure 5. If the latex-modified specimens that were wet-cured are allowed to dry eventually, the strength is restored. This shows that once the latex is allowed to undergo film formation, the basic strength will be achieved. After the film formation the standard moist-curing will not adversely affect the strength development. The strength increase will be similar to that of conventional concrete.

Figure 5 shows the effect of a 2-day moist, 5-day wet and 21 day dry curing on the compressive strength of latex-modified concretes (21). This confirms the earlier statement that optimum strength in latex-modified concretes is acheived when a reasonable degree of cement hydration at early ages under wet conditions is allowed. This should be followed by dry conditions to promote latex film formation. Immersion in water after dry curing causes a sharp reduction in the strength of all latex-modified systems (7). However the strength is recovered after subsequent drying (19).

COMPRESSIVE STRENGTH

Enhancement of compressive strength is not the main objective of adding latex to portland cement concretes. However, an increase in compressive strength may be achieved with the same workability because of the reduction in water-cement ratio that could be achieved with the addition of latex. For the same durability requirements, a reduced air content is adequate in latex-modified concretes, which consequently increases the strength. An increase in the antifoaming agent content (consequent decrease in air content) thus increases the compressive strength (19).

For the same cement content, the 3-day, 7-day and 28-day compressive strengths of latex-modified concretes reported by various investigators (16, 22, 23) are given in Table 5 (11).

Two studies (15, 16) were conducted on the effect of cylinder specimen size on compressive strength of LMC and both studies had shown that the influence of specimen size on compressive strength was negligible (Figure 6).

TENSILE AND FLEXURAL STRENGTHS

The addition of latex to portland cement concretes considerably increases the tensile and flexural strengths. This increase is due to the improvement in the cement hydrate-aggregate bond and due to the high tensile strength of the latex film.

Various factors such as the type and quantity of latex, cement, aggregates, water-cement ratio, curing methods, and testing procedures affect the flexural and tensile strengths.

DEFORMATION BEHAVIOR AND MODULUS OF ELASTICITY, DUCTILITY AND TOUGHNESS

The addition of latex in sufficient quantities will increase the ultimate strain capacity, the ductility and toughness of portland cement concretes. However there can be a significant, often undesirable, change in modulus of elasticity in compression for PVAC and a small change for PAE and SBR latexes. The increase in ductility and strain capacity depends on the latex-cement ratio. This can be very beneficial improvement when structures have to resist dynamic and impact loads as in the case of earthquake and blast resistant structures and machine foundations. The tensile stress versus strain relationships for various polymer-cement ratios are shown in Figure 7. The tensile strain at failure increased to 2 to 3 times higher than that of unmodified concretes. The modulus of elasticity in tension also decreases as the latex-cement ratio increases (24). The increased strain capacity is attributed to the polymer film effectively halting the propogation of microcracks.

The modulus of elasticity in compression and the poisson's ratio of latex-modified concretes are given in Table 6 (25). An increase in the polymer-cement ratio generally reduces the modulus of elasticity. The poisson's ratio of PAE and SBR-modified concretes are nearly equal to that of unmodified concrete regardless of the polymer-cement ratio. However for the PVAC-modified concrete the poisson's ratio increases with an increase in polymer-cement ratio. Two other studies (26, 27) had also reported similar findings. A significant reduction in modulus of elasticity was observed when the latex content exceeded 10 percent.

SHRINKAGE

Latex-modified concretes are more sensitive to initial plastic shrinkage if they are not properly cured. However, the drying shrinkage depends on the type and quantity of latex addition. The drying shrinkage increases with additional dry curing period. After 28 days of dry curing, the shrinkage almost ceases for all latex types and quantitites. The 28-day drying shrinkage generally tends to decrease with increasing latex-cement ratio. Some latexes (PVAC, NR and CR) when added cause a larger shrinkage than those of unmodified concretes. The drying shrinkage is less for concretes with higher latex-cement ratio because of higher water retention ability. It was reported by Kawano (28) that latex-modififed concretes had less shrinkage than conventional concretes mainly due to the effects of the surfactants and antifoaming agents added to the latexes.

The addition of SBR latex to concretes does not increase the drying shrinkage as reported by Michalyshin (29). In this study the drying shrinkage strains of latex-modified and conventional concretes with similar water-cement ratios were compared as shown in Figure 8 (Table 7).

In an earlier study (15), Ohama had obtained similar results. He investigated three latex-cement ratios and three different sizes of concrete specimens were used. The same workability was maintained by adjusting the water-cement ratio as shown in Table 8.

The measured shrinkage strains are given in Figures 9 and 10. He concluded that drying shrinkage does not depend on the latex content, but rather depends mainly on the water content. Comparing Figures 9 and 10, it can be seen that the drying shrinkage of SBR and EVA latex-modified concretes decreased with an increase in specimen size. The effect of the specimen size is almost the same for both unmodified concrete and latex-modified concrete.

CREEP

There is not much published information on the creep behavior of latex-modified concretes. An investigation by Ohama (30), has shown that the creep strains are significantly lower for latex-modified concretes than those of unmodified concretes for the same slump. He has also shown that the relationship between time (t) and creep strain (ϵ_C), creep coefficient (ϕ) (creep strain-elastic strain ratio) for latex-modified concretes could be expressed by the same expression that is generally used for unmodified concretes (Figures 11 and 12).

$$\epsilon_C \text{ or } \phi = \frac{t}{A + Bt}$$

Where A and B are empirical constants evaluated from experimental results. He attributes the low creep in latex-modified mortars due to the strengthening of the binders with polymer and the long term strength development with improved water retention.

IMPACT STRENGTH

Latex-modified concretes have higher impact strength than corresponding conventional concretes, because of the high impact resistance of latexes. The impact strength increases with an increase in latex-cement ratios.

BOND OR ADHESION STRENGTH

The incorporation of latex in the conventional concretes can dramatically improve the bond or adhesion strength to various substrates. This improvement depends on the type of latex and is attributed to the high adhesion property of the latex. Most latexes, when added to concretes, develop excellent adhesion to substrates like old concrete, stone, ceramic tile, brick, steel and wood. The bond strength depends mainly on the latex-cement ratio, the latex type, surface texture, temperature and humidity during placing, and conditions of the substrates to which the latex-modified concrete is bonded. The observed bond strength also depends on the test method used to measure this property.

Both tensile and flexural bond strengths depend to a very great extent on the moisture content of the concrete during testing. Adhesion is improved significantly in the dry state and there is a considerably less improvement in the wet state. This is a disadvantage of the latex-modified concretes, in that under wet service conditions the adhesion is reduced.

As stated above, the research conducted in ideal laboratory conditions has shown that adhesion is better when the substrate surface is dry. However, in the recommended guidelines for bridge deck overlay, it is required to place the overlay only on a wet surface. The surface should be in a saturated condition but without pools of water collecting on the surface. There are several reasons for this practical recommendation. When a thin overlay is placed on a dry surface, the dry surface will quickly absorb water from the overlay concrete and may induce plastic shrinkage cracks. Also the cooling effect due to the evaporation of water on the surface will have a beneficial effect on the overlay concrete; it will reduce temperature difference between the substrate and the overlay concrete, particularly when the overlay is placed in the summer months, and avoid thermal stresses between the overlay and the base concrete.

Recently a number of studies about adhesion of latex-modified concretes (10, 23, 26, 31 to 35) have been conducted in U.S.A. The adhesion in tension of latex-modified concrete has been measured using the tensile splitting test with unmodified prior-split concrete cylinder halves as substrate material (23). The latex-modified concrete cylinder specimens containing 15 percent latex-cement ratio and cured for 28 days failed at an average of 525 psi (3.62 MPa) tensile splitting strength. For all six specimens the failure surface was through the aggregate and no failure took place at the bonded interface. The direct tensile test method used (33) to measure the bond strength had shown that bond of latex-modified concretes exceeds 262 psi (1.81 MPa) at 3 days and 334 psi (2.30 MPa) at 28 days (Table 9). The SBR-latex-modified concretes had a significant increase in direct tension bond strength compared to that of the unmodified concrete.

Tests conducted at the Dow Chemical Company laboratories over a number of years (34) showed that the 28 day shear bond strength of SBR latex-modified concrete is almost twice that of conventional concrete. However, the actual test results have not been reported. A guillotine type device to shear a cap of latex-modified concrete off a cylinder of unmodified concrete was used in this test. The average values of bond strength for latex-modified concretes with 15 percent latex-cement ratio obtained from tests conducted over several years, were 256 psi (1.77 MPa) after a 7-day cure, and 469 psi (3.24 MPa) at 28 days.

The adhesion between reinforcing steel and the SBR latex-modified concrete has been found to be adequate to develop the yield strength of the steel (35). Using a newly developed test method for the determination of direct tension bond strength (31), Kuhlmann has shown that the bond of latex-modified concrete exceeds 70 psi (0.48 Mpa) in one day of curing at room temperature, 340 psi (2.35 Mpa) in 28 days and 450 psi (3.12 Mpa) in 90 days.

ABRASION RESISTANCE

Another improvement due to the addition of latexes to conventional concretes is the increase in their abrasion resisting ability. Many investigators (10, 36, 37, 38) have reported a tremendous increase in the abrasion resistance of latex-modified concretes for several types of latexes. Using a traffic simulator for abrasion test, Gierloff (38) showed that various PAE latex-modified concretes with a high latex-cement ratio and a low water-cement ratio resisted traffic abrasion very well.

In general the abrasion resistance of latex-modified concretes is influenced by the type of latex used, latex-cement ratio, and the wear conditions. An increase in latex-cement ratio increases the abrasion resistance and for SBR latex-modified concretes, the abrasion resistance is increased with an increase in bound styrene content.

THERMAL PERFORMANCE

The thermal expansion characteristics of latex-modified concretes are similar to those of unmodified concretes. They are mainly influenced by the type of aggregates used (39).

Most polymers undergo a rapid reduction in strength with increased temperature, particularly at temperatures higher than the glass transition temperature of the polymers (usually 80° to $100^{\circ}C$) and with higher latex-cement ratios. Most latex-modified concretes lose 50 percent or more of their strength at temperatures above $122^{\circ}F$ ($50^{\circ}C$) (7).

The incombustibility of latex-modified concretes is dependent on the type of latex used, latex content and latex-cement ratio. All latex-modified concretes with less than 5 percent latex-cement ratio have very good incombustibility. However the combustibility increases with increasing latex-cement ratio.

PORE STRUCTURE AND PERMEABILITY

The pore structure of the cement paste has the greatest influence on the permeability and hence the durability of the concrete. This indirectly affects the rate and amount of corrosion of reinforcement embedded in the concrete. The pore structure controls the rate of flow of chemicals and gases, including chloride ions, which cause the corrosion of steel in concrete. The pore structure is beneficially altered by the addition of latexes to cement paste. The micropores and voids normally occurring in portland cement systems are partially filled with the latex film that forms during curing. The filling of the pores depends on the types of latexes and the latex-cement ratios. The total porosity or pore volume is decreased with an increase in latex-cement ratio. Latex-modified concretes have fewer large radius pores (0.2 μm or more) and more smaller radius pores (75 nanometers or less) compared to unmodified concretes. This is responsible for the reduced permeability and water absorption of latex-modified cement systems. There are several tests such as water-vapor transmission, chloride permeability, water absorption, and carbonation resistance that measure the effect of pore structure modification.

Marusin (39), based on scanning electron microscopy investigations, observed that the conventional concrete and concretes modified with latexes showed a different distribution and shape of pores. While the large pores were surrounded by a number of smaller pores in conventional portland cement concrete, latex-modified concretes (both acrylic and S-B latexes) contained only single, mostly spherical-shaped, pores.

She also reported that the concrete weight changes due to immersing in a 15 percent NaCl water solution for 21 days and air-drying in a climate controlled room using 4 in. (10 cm) cubes. During the

21 days in the salt solution (soaking period) and the 21 days in the final drying period, the gain in weight and the weight loss at 3, 7, 14 and 21 days are shown in Figure 13. Conventional concrete with W/C ratio of 0.4 was able to expel about 65 percent of absorbed water by vapor transmission. Acrylic latex-modified concrete lost all the absorbed water and S-B latex modified lost 66 percent of its absorbed water when allowed to dry.

Several test methods have been devised to measure chloride permeability in concrete. The results of a FHWA study (16) to determine the chloride permeability from a 90 day ponding test are shown in Figure 14. Ohama (40) soaked cylinders in salt solutions for 28 and 91 days and measured the chloride penetration with an indicator solution on the concrete surface after the cylinders were split. The penetration decreased with an increase in latex content as shown in Figures 15 and 16.

For a quicker determination of the chloride-ion permeability, a Rapid Permeability Test (AASHTO T-277) was developed (41) and using this test several investigations have been conducted in the U.S.A., and Canada (17, 26, 32, 34, 42, 43). In latex-modified concretes, even at higher air contents, the air voids were small and well distributed and hence the permeability did not increase (17), as shown in Table 11.

The permeability is significantly reduced with age beyond the normal 28 day period (26, 42). Samples taken from field- placed latex-modified concretes at three different locations in the U.S., using different aggregates and cements, were cured in laboratory conditions (72° F and 50 percent relative humidity). The permeability results for these samples are shown in Figure 17. They all have low permeability at 90 days. In general, the latex-modified concretes are more impermeable than unmodified concretes. Figure 18 (34) shows the influence of latex content on the permeability of SBR latex-modified concrete. It is evident that latex-cement ratio below 10 percent does not reduce the permeability sufficiently and an increase in latex content above 15 percent does not change the permeability significantly. Therefore it seems that a latex-cement ratio of 15 percent is optimum with regard to permeability reduction in concretes.

Marusin (39) measured the absorbed chloride ion contents for acrylic and S-B latex modified concretes using 4 in. (10 cm) cube specimens. The chloride ion content of the drilled powder from the same specimen was determined using an acid digestion, potentiometric titration procedure. Both concretes with 25 percent latex and 2 to 4 percent air content contained about 0.02 and 0.03 percent chloride ion content in the depth interval of 1/2 to 1 inch (12 to 25 mm). The acrylic latex and S-B latex showed similar profiles at all depths. The performance of both latex modified concretes was almost similar in minimizing the ingress of chloride ions into these concretes and "both provided excellent performance" (39).

CARBONATION RESISTANCE

The carbonation resistance is an important property that influences the durability of the concrete and the corrosion of rebar. It has been reported that latex-modified concretes have superior carbonation resisting ability compared to unmodified concretes. The carbonation resistance is increased with an increase in latex-cement ratio. Ohama (44) investigated the latex-modified concretes exposed to carbon dioxide gas and carbon dioxide solution ($NAHCO_3$), under a pressure of 0.1 to 0.6 MPa (1 to 6 Kg/cm^2) up to a period of 6 months. After the exposure the samples were painted with phenolphthalein solution. The measured carbonation depths are shown in Figures 19 and 20. In both methods carbonation is significantly reduced by the addition of a latex to the concrete.

DURABILITY

Freeze-Thaw resistance - - In general, latex-modified concretes have good freeze-thaw resistance, provided they have been properly cured allowing the formation of the latex film. The freeze-thaw durability of the latex-modified concretes is improved even at a low polymer-cement ratio of 5 percent as compared to non-air entrained concrete. The freeze-thaw resistance is due to the combined effects of water impermeability and air-entrainment provided by the latex. An increase in latex-cement ratio may not increase the freeze-thaw durability.

Both laboratory and field studies (18, 26, 34, 43, 45, 46) conducted in the U.S. have shown that the latex-modified concretes have excellent freeze-thaw resistance. In the field, the latex-modified concretes placed late in the construction season (when freezing temperatures occurred before 28 days of curing) no apparant damage had occurred. This is attributed to the relatively dry conditions of the cool weather since latex-modified concrete cures by drying (11).

The durability factors for some latex-modified concretes obtained using the ASTM C666 freeze-thaw tests are given in Table 12 (34).

Resistance to deicing chemicals - - The resistance of concrete surfaces to scaling when exposed to deicing chemicals and freeze-thaw cycles is another measure of its durability. The scaling resistances of latex-modified and unmodified concretes, measured by the Indiana State Highway Commission using ASTM Test C-672, are given in Table 13 (34). Latex-modified concrete was given a rating '0' in a rating scale of 0 to 5, which indicated that no scaling had occured after 50 cycles of freezing and thawing.

Chemical Resistance

Generally, hydrated cement is non-resistant to inorganic or organic acids and sulfates. Most latex-modified concretes are damaged by these chemicals. However they may resist alkalies, fats, oils, and various salts except sulfates. Organic solvents easily attack most latex-modified concretes.

CURRENT USES AND POTENTIAL APPLICATIONS

The development of latex-modified concretes extended the use of these materials in applications where the conventional cement concretes could not be used effectively until now. Compared to conventional concretes, the latex-modified concretes can improve such properties like tensile and flexural strength, adhesion, water and air impermeability, abrasion resistance, toughness, and chemical resistance.

CURRENT APPLICATIONS

Latex-modified concretes are currently used in the following applications:

Overlays for bridge decks
Overlays for parking decks
Pavement centerline repair
Repair of buildings and other structures
Stadium repair, and
Pavement repair and patching.

The most important improvements in conventional concrete properties due to latex addition are reduced permeability to water and chloride ions, increased resistance to freeze-thaw action, and improved adhesion to old concrete substrates. Therefore it is a good material for overlays on bridges, parking garages, and exposed stadiums where the degree of reduction of chloride penetration (and hence corrosion elimination) is directly related to the overall performance and life of the structure.

The major current use of latex-modified concrete has been as overlays on bridge decks, which are typically 1.25 to 2 inch (30 to 50 mm) thick. These overlays have been applied to new construction as well as old bridges for repair and restoration. These overlays are intended to provide a well bonded, long-wearing riding surface and to act as a barrier to deicing salts. In the U.S.A.,thousands of bridge decks and parking garages have been rehabilitated since 1957 (5, 12, 16, 23, 26, 33, 45, 46, 48, 49, 50, 51).

A notable application of latex-modified concrete is in the repair of the Soldier Field Stadium in Chicago in 1981 (52). This 58 year old concrete stadium was structurally restored by bonding reinforced latex-modified concrete to the old concrete. Another notable application of latex-modified concrete was reported by Howard (53). The Marquam Bridge in Portland, Oregon, after 17 years of service, was overlayed in 1983 with latex-modified concrete with Type III cement which has given satisfactory results.

The first time application of high early strength latex-modified concrete overlay for the Virginia Department of Transportation was reported by Sprinkel (12, 13). The overlay was prepared with more cement (type III) and less water than is used in conventional latex-modified concrete overlays. The traffic was allowed after 24 hours rather than the 4 to 7 days required for conventional latex-modified concrete. After 1 year of service, tests of the bond strength and penetration to chloride ion indicated that the overlay was performing satisfactorily. This application technique, if it continues to give satisfactory results in the future, will be a boon to the highway repair and rehabilitation projects as well as new construction. It could be used in situations in which it is desirable to accelerate construction, to reduce inconvenience to motorists, to allow for installation during off-peak traffic periods such as weekends, to provide a more rapid cure in cold weather, to provide low permeability and to provide high strength at a very early age.

Some of the other high-early strength latex-modified concrete overlays are: a 150 foot-long section installed on the Delaware Memorial Bridge in 1987 (traffic was allowed on this overlay 24 hours later); northbound I-81 in Saltville, Virginia (1985); and the Chicago Skyway I-90 at 79th street in Chicago in 1987. All these overlays are performing well.

SPECIAL APPLICATIONS OF LATEX-MODIFIED CONCRETES

In Japan (54), an experimental investigation was conducted to demonstrate the advantage of higher ductility of the latex-modified concrete, compared to conventional concrete, to resist seismic loads. The experiments were conducted on ductile-latex-modified concrete shear walls.

Latex-modified systems had been used to produce foam lightweight concrete (55) which had a higher strength, higher toughness and lower permeability than ordinary foam concrete.

POTENTIAL APPLICATIONS FOR LATEX-MODIFIED CONCRETES

Use in Fiber Reinforced Concrete: The addition of steel fibers to concrete considerably improves the flexural strength, impact strength, direct tensile strength, shock resistance, ductility, failure toughness and post-crack load carrying capacity. It had been demonstrated that these improvements could be achieved without a reduction of workability and this material is very suitable for bridge deck overlays, new pavements and for the construction of other concrete structures (56, 57). When latex is added to the steel fiber reinforced concrete, tremendous improvements could be achieved in the impermeability, freeze-thaw resistance, deformation characteristics, elastic and mechanical properties (58). The presence of latex in the matrix of the concrete composite has a higher synergestic effect than the ordinary fiber reinforced concrete. Steel, glass, polymers and carbon fibers could be used for the purpose of increasing the toughness and flexural strength of latex-modified concrete, or latex may be added to fiber reinforced concretes to improve their workability, to reduce the drying shrinkage, to improve the durability and weatherability, to increase the bond strength, and to increase the resistance to carbonation, chloride ion and oxygen penetration. Carbon fiber reinforced polymer-modified mortar have been developed (59) and employed in concrete repairs. The presence of polymer latex in fiber reinforced mortar could increase the strength values by a factor of four, compared to the plain cement mortar. They could induce changes in the load-deflection curves, making them smoother without discontinuities. There will be beneficial changes in the microcracking characteristics. Elastomeric latexes might perform better than thermoplastic latexes.

Thus, the marriage between fibers and the latex-modified concretes could produce a well suited material for the rehabilitation of bridge decks, and parking garages, repair and restoration of older structures, and new construction of earthquake and blast resistant structures. There is a good scope for someone to develop thin precast elements or slabs with fiber reinforced latex-modified concretes for bridge deck and parking garage overlays and for other repair works or new constructions where durability and ductility are the primary concerns.

The first reported (60) use of latex-modified steel fiber reinforced concrete was in the Morrison Bridge Deck repair in Portland, Oregon. This overlay reduced costs and overlay thickness for the bridge and also contributed significantly to the structural strength of the existing deck. The inclusion of latex reduced the water demand of the mixture, thus helping to control excessive shrinkage and reduce the permeability, and increasing the resistance to penetration by deicing salts (61). Other advantages of latex inclusion were a very fluid concrete mixture and better bonding between the overlay and the existing deck. The overlay has been in place since 1980 with no deterioration of the deck surface.

The same contractor who carried out the Morrison Bridge Deck overlay also installed similar overlays on the Sandial-Sandy River Bridge at Troutdale, Oregon in 1982 and the Centralic, Washington Bridge in 1983. The former was a restoration project for a deteriorated bridge deck, the latter a new bridge where some structural problems apparently developed during construction (62). The Oregon Department of Transportation had subsequently approved similar overlays for three other bridge deck repair projects on I-80 near Troutdale Oregon (62).

In 1981 Lankard (61) carried out a comparative evaluation of latex-modified and various steel fiber reinforced latex-modified concrete mixtures intended for use in overlay applications in Ohio. As a result of this successful performance of the latex-modified steel fiber reinforced concrete, it was used for overlays on a bridge in Ohio, State Route 665. One year later it was reported to be in excellent condition (62).

Use in Conventional Shotcrete

Another area of potential application for latexes is conventional shotcrete. Many shotcrete applications may need a protective membrane or matrix which is impermeable and will resist corrosion and freeze-thaw damage. Latexes could be added to shotcretes to achieve such a protection. In 1973, investigations of structures containing polymers were reported by Graham (63) and Bates (64). In 1984, Chynoweth (65) advocated strongly the use of latex-modified shotcretes. In a few occasions latex-modified shotcretes have been used (66). The inclusion of a latex into a shotcrete mixture results in the development of a latex binder throughout the shotcrete and this imparts increased bond, flexural and tensile strength, and improved ductility and toughness. As in the case of concrete, the bridging action of the polymer binder across micro cracks in the cement paste coupled with the continuous latex film throughout the shotcrete matrix results in a reduced permeability to water and corrosive solutions. If properly done, bonding and reduced shrinkage make the latex-modified shotcrete a more compatible repair material than conventional shotcrete. Latex-modified shotcrete could be more appropriately used in repair of structures subjected to wet-dry cycles and high concentration of chlorides, such as bridges and waterfront structures, especially along the coastal regions.

A failure of a latex-modified shotcrete application has caused some concern about the suitability of the material. The possibility of a special type of bond failure in styrene-butadiene latex-modified concrete was pointed out by Marusin (66). Polymer-rich films were noticed on the interface surfaces of shotcrete layers due to the latex migration "to form films that hardened on the surface of previously applied layers". It was concluded that extensive cracking of the latex-modified, dry-mixture shotcrete structures occurred as a result of both separation between the shotcrete layers and early shrinkage. "Applying new shotcrete layers over the hard surface of previously applied shotcrete that was partially covered with a smooth, dense, and nonabsorptive polymer-rich film caused by latex migration" caused the delamination of shotcrete layers.

In another extensive study of the same failure, involving field tests and laboratory simulations of field failure mechanism, it was concluded that improper shotcreting procedures had created interfacial zones of high porosity and poor bond between successive shotcrete layers. The layers of porous overspray, entrapped rebound and hardened SBR latex film were not removed before applying another layer. It was believed that the drying shrinkage and curling stresses on layers of unreinforced shotcrete, which already had very weak interfacial bond, had precipitated the failure of the repaired shotcrete. It was not a failure of the latex-modified shotcrete as a material. The adoption of proper and appropriate procedures and safety precautions could have averted this reported failure. Adequate training of the construction personnel may be needed to expand the application of latex-modified shotcrete for which there is a great potential and existing need.

Use in Fiber Reinforced Shotcrete

As stated in the case of fiber reinforced concrete, the addition of latex might produce a suitable and efficient material for the repair and rehabilitation of certain types of structures and for the new construction of particular structures. Steel fiber reinforced shotcrete was first introduced in U.S.A. in the 1970's (67) and since then it has been used in numerous applications, mainly in new construction, or lining rock slopes and underground openings in mines and tunnels.

Latex-modified steel fiber shotcrete will have excellent bond characteristics, very high flexural strength, toughness, impact strength, fatigue strength, lower shrinkage, and durability. This innovative material will be very attractive as a rehabilitation material and will provide a viable alternative to conventional rehabilitation procedures. The potential applications where latex-modified fiber shotcrete may provide an economical and technically feasible alternative to conventional rehabilitation procedures include the following: repair of corrosion and impact-damaged structures such as bridge deck soffits, girders, and abutments and marine structures such as surge breakers, jetties, sea walls, dolosse, and piles; rehabilitation of deteriorated tank linings, drains and trenches.

Use in Foundations and Structures Subjected to Dynamic Loads

Toughness, ductility, and high energy absorption capacity are essential requirements for structures likely to be subjected to suddenly applied loads like earthquakes, blasts, tornados, and wind gusts. The same is true for structures subjected to dynamic loads or repeated loads, like machinery foundations. The required ductility can be induced in concrete by the addition of an appropriate quantity of latex. As stated earlier, latex-modified concretes have markedly increased deformation characteristics and ductility. However this has to be achieved without drastically reducing the strength properties. Experimental investigation using concrete shear-wall structures has demonstrated this feasibility (54). A latex-modified concrete with 25 percent polymer-cement ratio had nearly twice the toughness of unmodified concrete.

Researchers in India (68) have demonstrated that in natural rubber latex-modified concretes, pronounced improvements in deformation characteristics could be achieved without sacrificing the strength properties. This was done with proper combination of natural rubber latexes, chemical admixtures and superplasticizers. The potential application of latex-modified concrete for structures subjected to dynamic loads, like machine foundations, has been demonstrated (69). Experiments were conducted using ordinary concrete and latex-modified concrete footings of three different thicknesses for three static loads at four excitation levels. Experimental results revealed that the amplitude of resonance was reduced considerably in the latex-modified concrete foundations. It was also observed that the damping factor of the latex-foundation-soil system was considerably larger than that for ordinary concrete foundations. Therefore, it was stated that the use of latex-modified concrete in foundations could lead to more economical designs.

Use in Marine and Offshore Structures

Latex-modified concrete, with its tremendously reduced permeability to water and salt solutions, enhanced resistance to carbonation and corrosion, increased flexural strength and ultimate strain capacity, is a well suited candidate for use in marine structures and offshore structures.

At Florida Atlantic University, an investigation (70) was conducted to evaluate the suitability of latex-modified fiber reinforced concretes for marine structural applications. Fiber (steel, glass and polymer) and latex-modified concrete structural members were exposed for varying periods to marine environments. The structural integrity (the compressive, tensile, flexural, and impact strengths over an extended period of time) and long-term durability were studied with alternate exposure to sea water and oven drying. The findings from this limited study had indicated the superior behavior of latex and fiber modified concretes in the marine environment. Polymer fibers were more stable in the marine environment compared to glass and steel. However the addition of latex had improved the durability characteristics in all concretes. The investigation concluded that the latex fiber-modified concrete is the most suitable for long term applications in the marine environment.

Miscellaneous Potential Applications

Roller compacted concrete is used in the construction of dams and it could be also used in highway and airport pavements and parking lots. Lack of adequate freeze-thaw durability is a major problem in the use of roller compacted concrete (71). The addition of a latex to roller compacted concrete could improve its durability. However, cost may be a limiting criterion.

Latexes or latex-coated aggregates may be used in critical situations to control or to eliminate the alkali-aggregate reactions and long term alkali-silica reaction of the concrete (72).

Small amounts of latexes could be added to conventional shotcrete for the purpose of controlling dust and reducing rebound rate during shotcreting (73).

Artificial wood with properties similar to wood was developed using calcium silicate, polymer dispersion and glass fibers without portland cement (74).

Latexes may be increasingly used in precast and prestressing industry. Mass production of large anticorrosive structural elements and weather-proofed roof-decks could be economical and they may be long-lasting.

Use To Improve Skid Resistance

The loss of skid resistance from traffic due to the polishing of soft aggregates will severely restrict the useful life of a bridge deck. Such damaged bridge decks can be rehabilitated by utilizing the bonding capabilities of latex-modification. Sprinkel has reported the application of a latex-modified portland cement slag slurry in Virginia in 1989 and 1990 to increase the skid resistance of pavements and decks constructed with insufficient microtexture and macrotexture (75). This technique had been used only twice before, in Indiana in 1980 and in San Juan, Puerto Rico in 1986.

Two experimental applications, one a 300 ft. (100 m) section of pavement in the travel lane, (May 1989), and another a passing lane and travel lane of a bridge on I-81 in Rockbridge County (November 1969 and April 1990), were described. In two of the three cases the traffic was allowed after about 48 hours of cure. The initial cost of the latex-modified slag slurry was \$5.90 /yd^2, which is low compared to the \$35 /yd^2 cost of a 1.3 inch (33 mm) minimum thickness latex-modified concrete overlay. It was concluded that "the application of a latex-modified portland cement slag slurry is an economical technique for increasing the microtexture and macrotexture of hardened concrete surfaces constructed with polishing or nonpolishing aggregates. The technique also increases the cover over the rebar" (75).

Use of Silica-Fume to Enhance the Properties of Latex-Modified Systems

Silica-fume is being added to conventional concretes to highly increase their strength properties and to tremendously reduce their permeability. Latexes are added to portland cement concretes to primarily reduce their permeability and to increase their adhesive strength. Walters (76) reported an experimental investigation of the combined use of silica fume and styrene-butadiene latex in portland cement concretes. Silica-fume-latex-modified concretes had improved properties compared to those using one or the other of the admixtures alone. There was no need for any modification of the currently used equipment in placing, consolidating and finishing the silica-fume-latex-modified concretes.

Effect of Fly Ash on Latex Modified Concrete

The compatibility of fly ash with the SBR latex-modified concrete was reported by Diamond (77). Concretes batched with SBR latex and incorporating 15% and 25% of fly ash did not have any adverse effects. Four different Indiana fly ashes, including one class C and three class F ashes of varying properties, were used in the investigation. The fly ashes did not seriously degrade the development of strength in the latex-modified concretes. Some reduced chloride permeability was observed. Different fly ashes affected differently the performance of latex-modified concrete, the particular class C ash used showed generally superior effectiveness. However the differences did not appear to be of great practical importance. The differences between the results of 15% and 25% fly ash treatment levels for a given fly ash were not very great.

OBSERVATIONS AND CONCLUSIONS

Based on an intensive and objective review of the cited references, and discussions held with LMC researchers from various countries, and users (mainly State Highway Engineers), the following conclusions were made.

1. For the past 30 years considerable research and development of LMC had been conducted mainly in the U.S.A and Japan with some minor work in Europe and India. In the U.S.A, most of the development work originated from the latex producers and the research work was mainly applications oriented.

2. Property enhancements of conventional portland cement concretes are possible with the addition of appropriate type and quantity of latexes. These modified concretes have high adhesive (bond) strength, higher tensile and flexural strengths, low permeability to water, air, gases and chemicals, better durability and weatherability, higher carbonation resistance, better corrosion protection for the steel reinforcement, higher ductility, higher toughness, higher strain capacity in precracking and postcracking stages, lower shrinkage, higher impact strength, better abrasion resistance and better skid resistance. Other benefits include less bleeding and segregation and better water retention in the fresh concrete.

3. Properly designed latex-modified concretes have been proven to be safe, durable, beneficial, and economical, if they are wisely selected and properly used. They are suitable for various applications and have a high potential in repair and rehabilitation projects.

4. In spite of the fact that LMC had been used for the past 50 years, currently there are no standard ASTM, AASHTO or ACI specifications for quality control and application of LMC. ACI Committee 548 will be publishing shortly a standard for the construction of bridge deck overlays with styrene-butadiene latex modified concrete. In contrast, there are 17 Japanese Industrial Standards (JIS) for quality and testing methods of polymer modified mortars and polyester concretes. Ten of these deal with latex modified mortars.

5. Styrene-butadiene latex has the longest track record of successful applications and it is currently used much more than other equally promising latexes. In the U.S.A., LMC is primarily used for bridge deck overlays and to a minor extent for parking garage overlays. It is used both in new construction and repair of deteriorated decks.

6. New applications are being explored for profitably and efficiently using the latexes. These new and potential applications include the addition of latexes in fiber reinforced concrete, conventional shotcrete, fiber reinforced shotcrete, roller compacted concrete, lightweight foam concrete, for controlling or eliminating the alkali-aggregate, and alkali-silica reactions, to improve skid resistance, and to enhance the properties of silica fume and fly ash concretes.

Suggested new structural applications include foundations and structures subjected to dynamic loads, marine and offshore structures, mass production of large anti-corrosive structural elements and weather-proofed roof decks.

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TABLE 1 QUALITY REQUIREMENTS FOR LATEXES SPECIFIED IN JIS A 6203

Type of Test	Evaluation	Requirement
Latex test	Appearence	Exclusive of coarse particles foreign substances and coagulants.
	Total solids	Not less than 35.0% and within ±1.0% of the value marked by the manufacturer.
Polymer-Modified Mortar test	Flexural strength	Not less than 562 psi
	Comp. strength	Not less than 1420psi
	Adhesion	Not less than 142 psi
	Water Absorption	Not more than 15.0%
	Amount of water permeation	Not more than 0.66 lbs (30g)
	Length change	0 to 0.150%

1 psi = .00695 Mpa.

TABLE 2 TYPICAL PROPORTIONS FOR SBR LATEX-MODIFIED CONCRETE MIXTURES ACI COMMITTEE 548A (FROM REF.11)

Cement	685 lbs (311 kg)
Sand	1710 lbs (776 kg)
Coarse Aggregate	1140 lbs (517 kg)
Latex(48% solids)	24.5 gal (84.4 litres)
Water	19.0 gal (17.3 litres)

TABLE 3 COMPARISON OF MIXTURE PROPORTIONS FOR HIGH EARLY STRENGTH LATEX MODIFIED CONCRETE OVERLAY AND OTHER BRIDGE DECK CONCRETES USED IN VIRGINIA (FROM REF.12)

Material	Conventional concrete	LMC	LMC-HE
Cement, lb./cu.yd.	635	658	815
Water-cement ratio	0.45	0.37	0.34
Latex, gal./bag	0	3.5	3.0
Air, %	5 to 8	3 to 7	3 to7
Fine agg.(lb/cu.yd)	1,178	1,571	1,402
Coarse agg.(lb/cu.yd)	1,809	1,234	1,142

Note: Fine agg. specific gravity was 2.61, fineness modulus 3.0, Coarse agg. specific gravity was 2.51.
LMC : Latex modified concrete
LMC-HE: High early strength Latex modified concrete
1 lb./cu.yd. = 0.593 kg/m^3; 1 gal. = 3.785 litres

TABLE 4 MIX PROPORTIONS OF LATEX-MODIFIED CONCRETES FOR CONSTANT WORKABILITY (FROM REF. 15)

Type of Concrete	Cement Content lb/cu.yd (kg/m^3)	Latex Solids-cement ratio(%)	Water-Cement ratio	Sand-Aggregate ratio	Slump Inch (mm)
Unmodified	507 (300)	0	.675	.45	6.3 (160)
SBR modified	507 (300)	5	.583	.45	6.3 (160)
		10	.503	.45	6.1 (155)
		20	.410	.45	6.3 (160)
EVA modified	507 (300)	5	.587	.45	6.1 (155)
		10	.523	.45	6.1 (155)
		20	.443	.45	6.1 (155)

TABLE 5 COMPRESSIVE STRENGTH OF LATEX-MODIFIED CONCRETES (FROM REF. 11)

Ref.	Cement bags/yd^3	W/C ratio	Air (%)	slump in(cm)	Compressive psi (Mpa) 3-day	7-day	28-day
16	7.0	0.31	2.1	3.5(9)	N.R.	N.R.	5245 (36.2)
22	7.0	N.R.	3.9	4.7(12)	3400 (23.9)	4100 (28.3)	5620 (38.7)
23	7.0	0.37	4.0	3.9(10)	N.R.	4025 (27.8)	5005 (34.5)

Note: N.R. - Not Reported.
Ref. - Reference.

Table 6 MODULUS OF ELASTICITY AND POISSON'S RATIO OF LATEX - MODIFIED CONCRETES (FROM REF.25)

Type of Concrete	Polymer-Cement (%)	Modulus of Elasticity in Compression ($\times 10^5$ kg/cm^2)	Poisson's Ratio
Unmodified	0	2.11	0.17
PAE modified	5	2.27	0.16
	10	2.36	0.17
	15	2.30	0.17
	20	2.24	0.17
SBR modified	5	2.28	0.16
	10	2.43	0.18
	15	2.42	0.18
	20	2.02	0.18
PVAC modified	5	1.90	0.16
	10	1.79	0.19
	15	1.35	0.24
	20	1.00	0.29

1 Kg/cm^2 = 0.0981 MPa

TABLE 7 MIXTURE PROPORTIONS FOR CONCRETE USED IN LINEAR SHRINKAGE STUDY- FIGURE 8 (FROM REF. 11)

Type	Slump Inch(Cm)	WR %	AEA %	Air %	Water-Cement ratio	Compressive Strength at 28-day, psi(Mpa)
LMC	5.5(14)	-	-	5.0	0.33	6005(41.5)
	7.9(20)	-	-	4.7	0.37	5510(38.1)
	9.8(25)	-	-	3.7	0.42	5210(36.0)
Conventional*	1.6(4)	.42	.05	9.2	0.42	5170(35.7)

Type I cement used for all concretes.

* Conventional mixture contained a water reducer (WR) and air entraining agent (AEA). Values gieven are by weight based on Cement.

LMC - Latex-Modified Concrete

All mixtures had fine/coarse aggregate of 1.5/1.0, and cement factor of 658 lb./cu. yd. Latex solids/cement of latex-modified concretes was 0.15.

TABLE 8 MIXTURE PROPORTIONS OF CONCRETES USED IN SHRINKAGE STUDY-FIGURES 9 & 10 (FROM REF. 15)

Type of Concrete	Cement Content lb/yd(kg/m^3)	Latex Solids-Cement	W/C ratio	Fine Coarse Agg. ratio	Slump in(cm)
Unmodified	505(300)	0	0.67	0.45	6.3(16.0)
Latex-Modified	505(300)	0.05	0.58	0.45	6.3(16.0)
		0.10	0.50	0.45	6.1(15.5)
		0.20	0.41		6.3(16.0)

TABLE 9 TENSILE BOND STRENGTH (FROM REF. 19)

Cure days	Unmodified Psi (MPa)	Latex-Modified Psi (MPa)
3	161 (1.11)	262 (1.81)
7	181 (1.25)	278 (1.92)
14	215 (1.48)	327 (2.26)
28	243 (1.68)	334 (2.30)
90	256 (1.77)	365 (2.52)

TABLE 10 CHLORIDE ION DIFFUSION COEFFICIENT OF LATEX-MODIFIED CONCRETES (FROM REF.7)

Type of concrete	Polymer-cement ratio,%	Apparent chloride ion diffusion coefficient cm2/s
Un-modified	0	2.2×10^{-8}
SBR-modified	10 20	1.9×10^{-8} 9.3×10^{-9}
EVA-modified	10 20	7.9×10^{-9} 1.0×10^{-8}
PAE-modified	10 20	6.2×10^{-9} 5.8×10^{-9}

TABLE 11 TOTAL COULOMBS PASSED FOR LATEX MODIFIED CONCRETE HAVING VARIOUS AIR CONTENTS (FROM REF.17)

Mix	Air Content %	Age days	Total Coulombs
1	3.0	63	650
		69	740
2	4.5	28	520
		35	455
		91	240
3	5.6	28	935
		29	870
4	7.5	16	1105
		24	835
		63	530
		70	780
5	12	41	760
		50	510
6	15	35	705
		37	650
		91	425

TABLE 12 FREEZE-THAW RESISTANCE OF LATEX-MODIFIED CONCRETES (FROM REF.34)

Air Content (%)	Cure	No. of Cycles	Durability Factor
5.1	1-day wet 27-day dry	323	91
5.8	1-day wet 27-day dry	323	98
5.2	1-day wet	323	91

TABLE 13 SCALING RESISTANCE OF LATEX-MODIFIED CONCRETES (FROM REF.34)

	Air Content (%)	No. of Cycles	Rating
Latex	4.1	50	0
Conventional	4.9	50	2

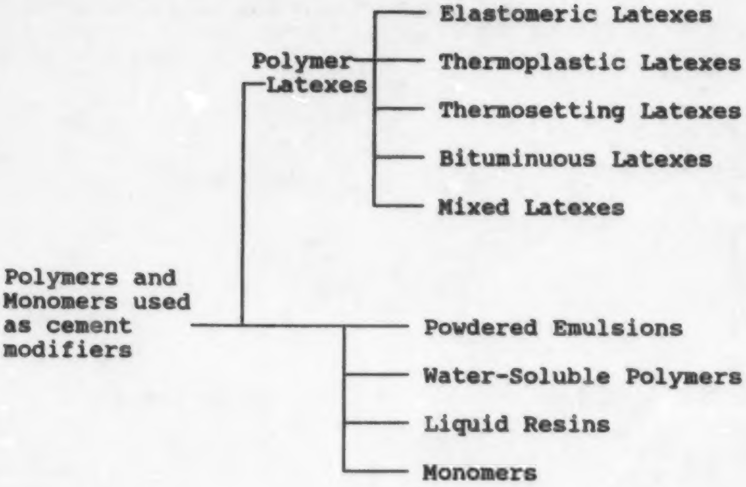
Elastomeric Latexes
Polymer Latexes
Thermoplastic Latexes
Thermosetting Latexes
Bituminuous Latexes
Mixed Latexes
Polymers and Monomers used as cement modifiers
Powdered Emulsions
Water-Soluble Polymers
Liquid Resins
Monomers

FIGURE 1. POLYMERS AND MONOMERS USED AS CEMENT MODIFIERS (FROM REF.7)

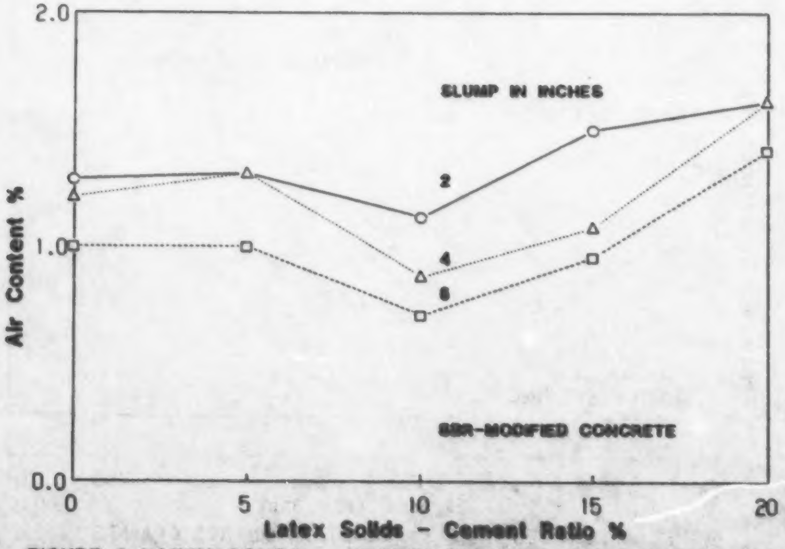

FIGURE 2 LATEX SOLIDS - CEMENT RATIO Vs AIR CONTENT FOR SBR LATEX MODIFIED CONCRETES (FROM REF.14)

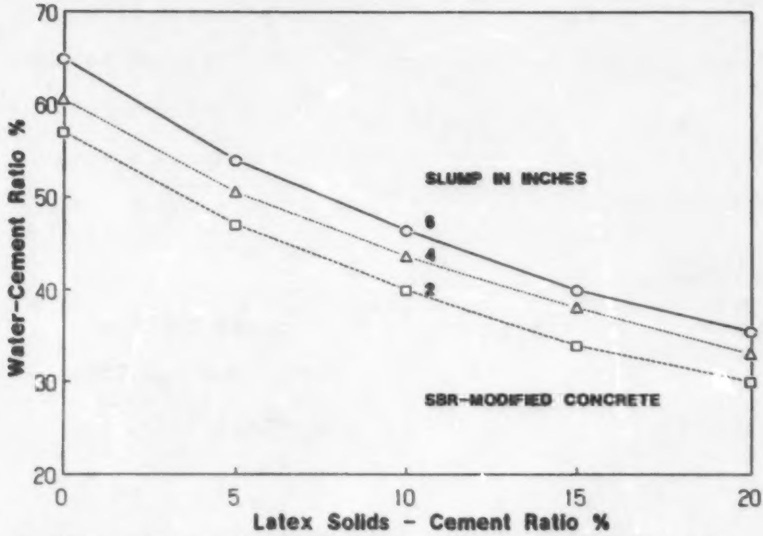

FIGURE 3 RELATION BETWEEN LATEX SOLIDS - CEMENT RATIO AND WATER-CEMENT RATIO FOR CONSTANT SLUMPS (FROM REF.7)

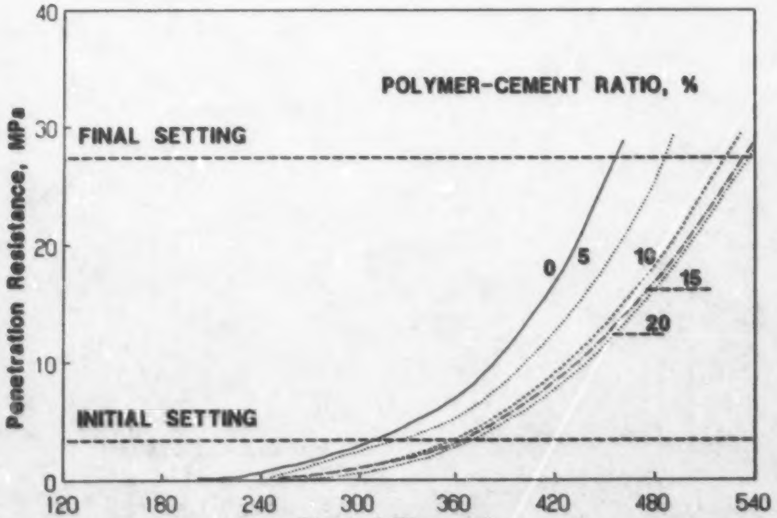

FIGURE 4 ELAPSED TIME Vs PENETRATION RESISTANCE OF SBR-MODIFIED CONCRETE (FROM REF.14)

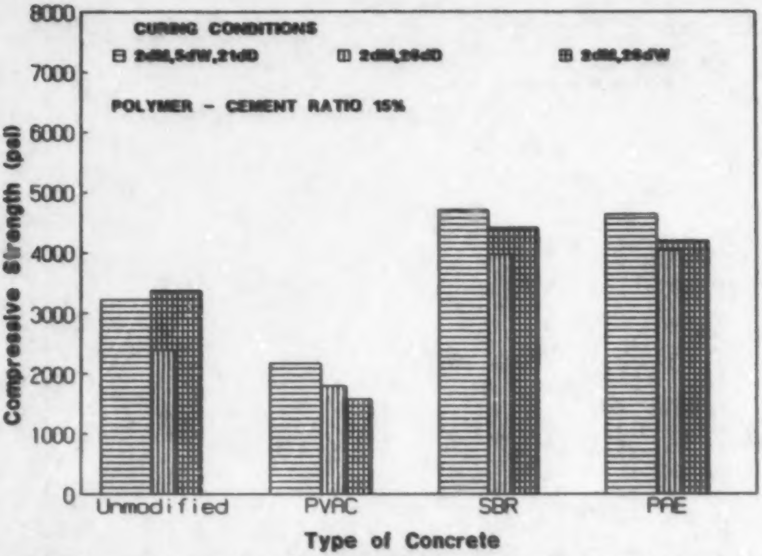

FIGURE5 EFFECT OF CURING CONDITIONS ON COMPRESSIVE STRENGTH OF UNMODIFIED Vs LATEX-MODIFIED CONCRETES (FROM REF.21)

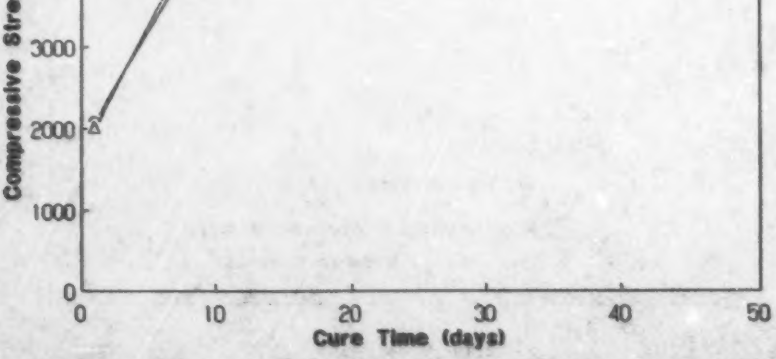

FIGURE 6 EFFECT OF CYLINDER SPECIMEN SIZE ON COMPRESSIVE STRENGTH OF LATEX-MODIFIED CONCRETES (FROM REF.16)

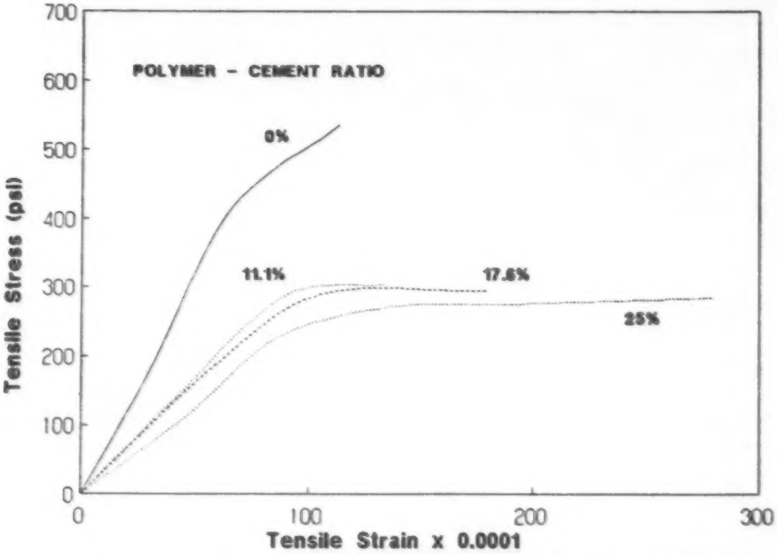

FIGURE 7 TENSILE STRESS Vs STRAIN FOR SBR-MODIFIED CONCRETES (FORM REF.24)

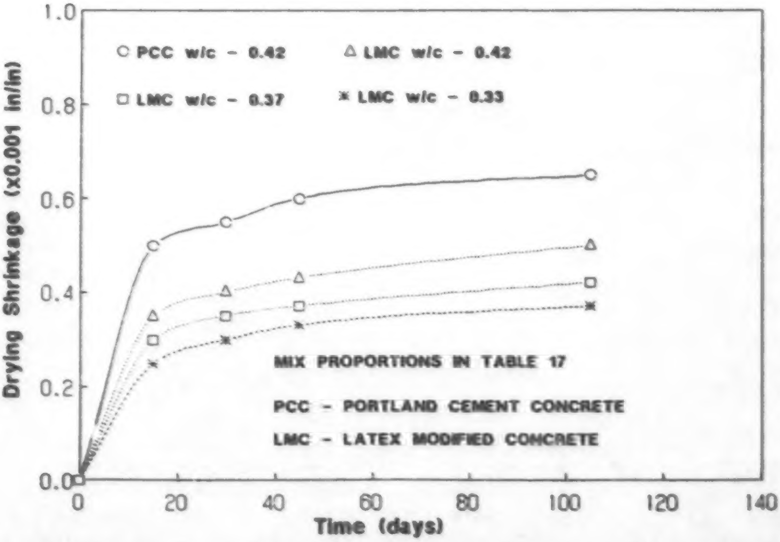

FIGURE 8 DRYING SHRINKAGE FOR UNMODIFIED Vs LATEX-MODIFIED CONCRETES (FROM REF.29)

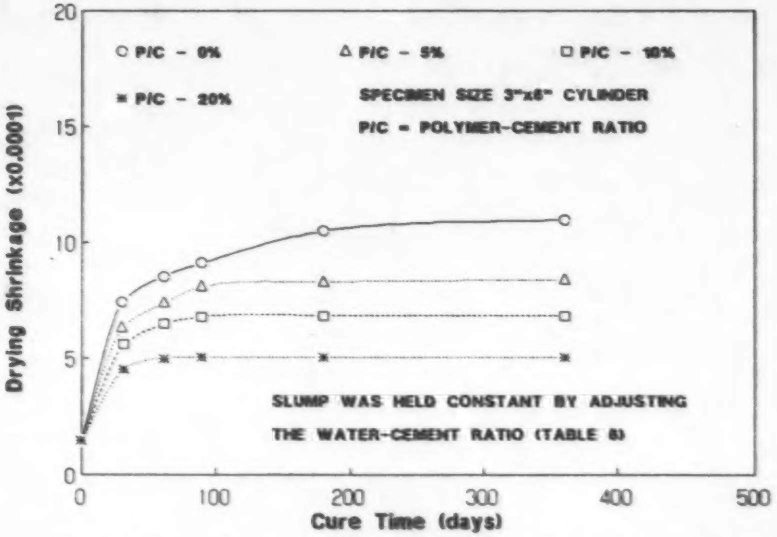

FIGURE 9 DRYING SHRINKAGE Vs TIME FOR 3" X 6" CONCRETE CYLINDERS (FROM REF.15)

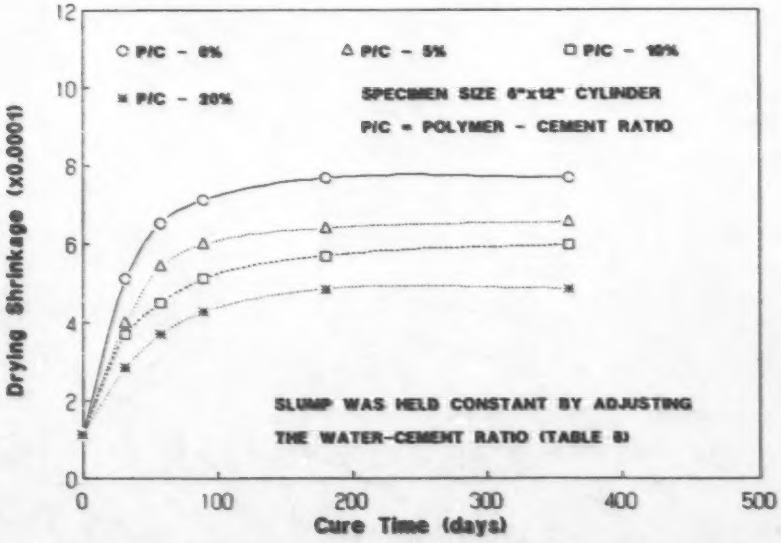

FIGURE 10 DRYING SHRINKAGE Vs TIME FOR 6" X 12" CONCRETE CYLINDERS (FROM REF.15)

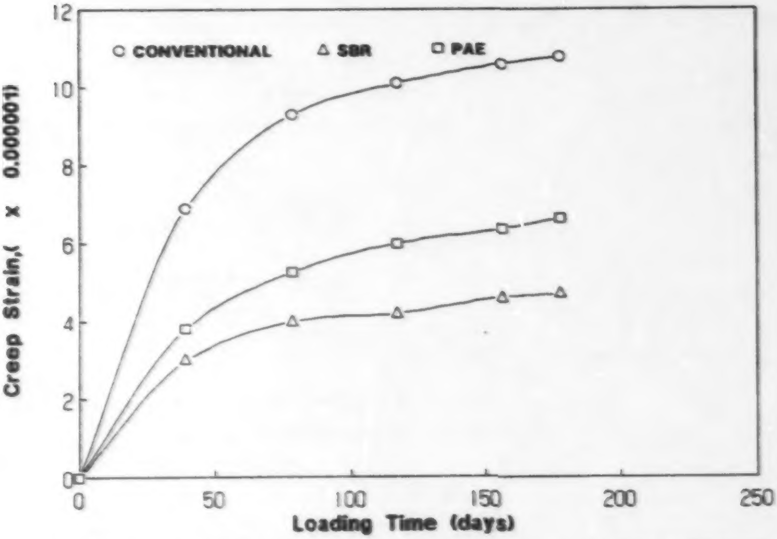

FIGURE 11 CREEP STRAINS OF UNMODIFIED Vs LATEX-MODIFIED CONCRETES (FROM REF.30)

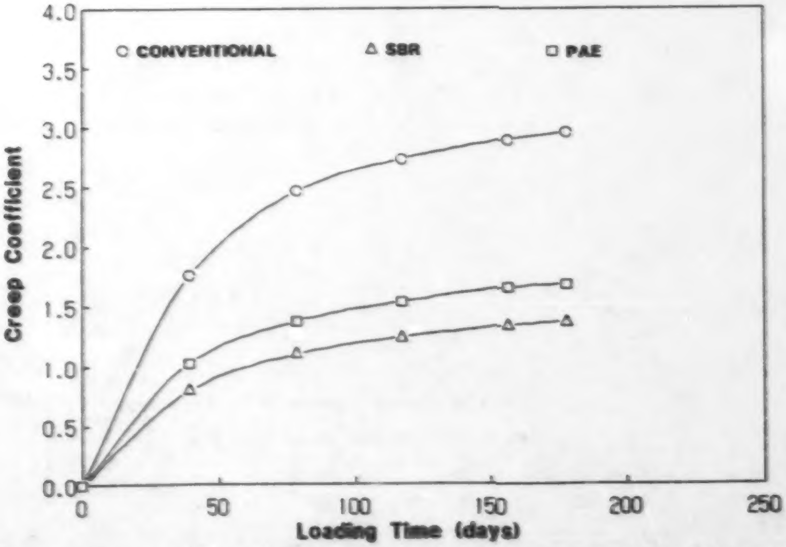

FIGURE 12 CREEP COEFFICIENT Vs LOADING TIME FOR UNMODIFIED AND LATEX-MODIFIED CONCRETES (FROM REF.30)

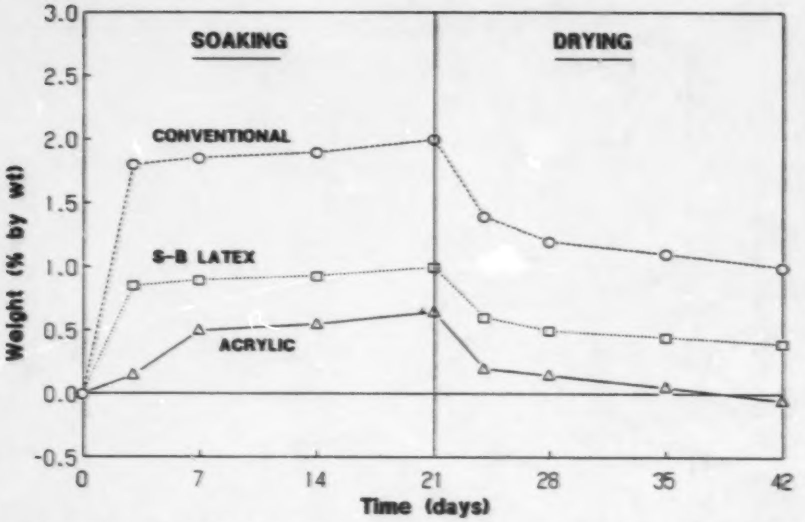

FIGURE 13 WEIGHT GAIN AND LOSS FOR LATEX-MODIFIED CONCRETES (FROM REF.39)

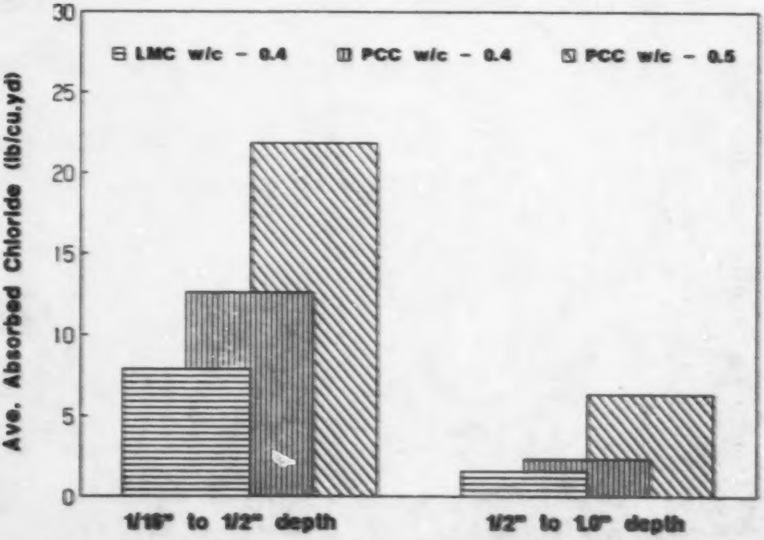

FIGURE 14 CHLORIDE PERMEABILITY BY 90-DAY PONDING TEST FOR UNMODIFIED Vs SBR LATEX-MODIFIED CONCRETES (FROM REF.16)

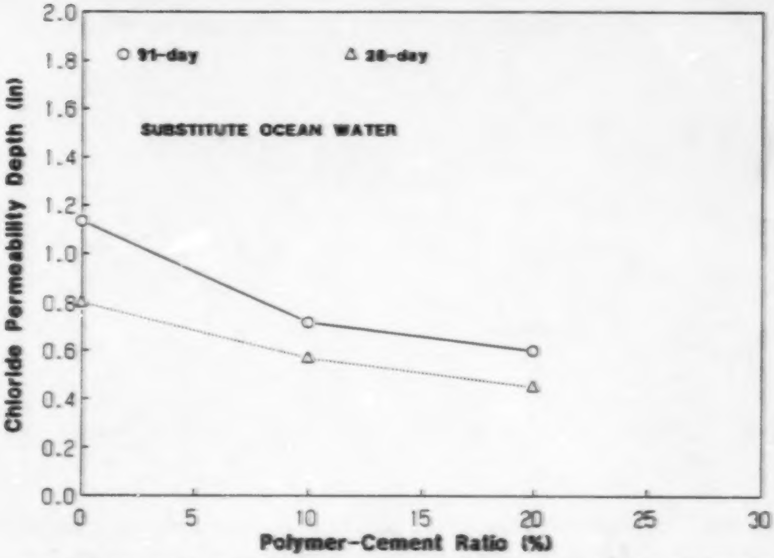

FIGURE 15 CHLORIDE PERMEABILITY DEPTH Vs POLYMER-CONTENT RATIO FOR CONCRETE CYLINDER SPECIMENS (FROM REF.40)

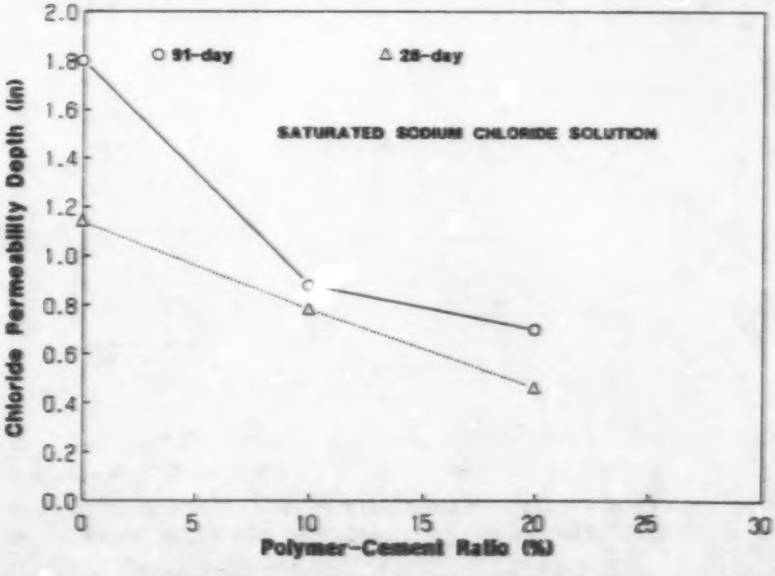

FIGURE 16 CHLORIDE PERMEABILITY DEPTH Vs POLYMER-CONTENT RATIO FOR CONCRETE CYLINDER SPECIMENS (FROM REF.40)

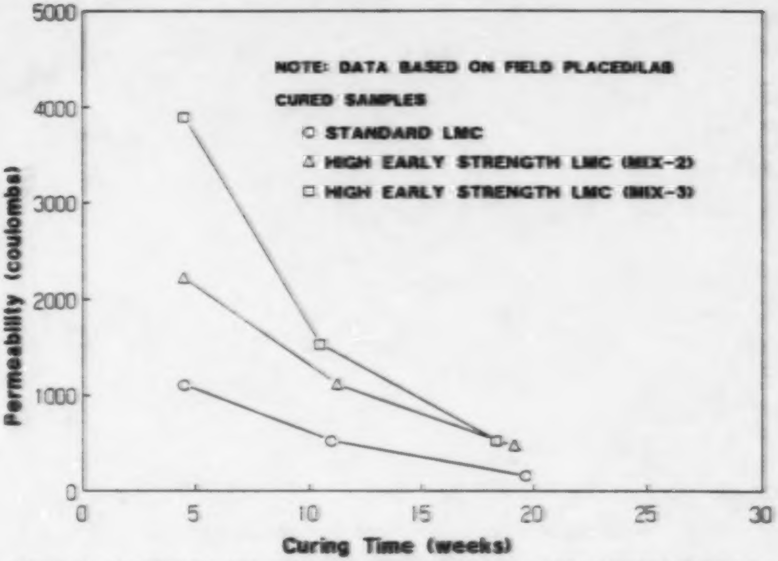

FIGURE 17 EFFECT OF AGE ON PERMEABILITY FOR LATEX-MODIFIED CONCRETES (FROM REF.26)

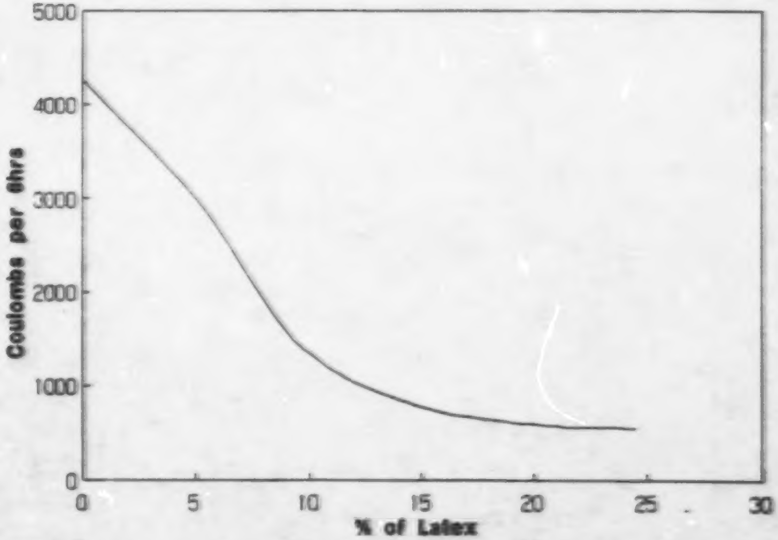

FIGURE 18 EFFECT OF LATEX LEVEL ON CONCRETE PERMEABILITY (FROM REF.34)

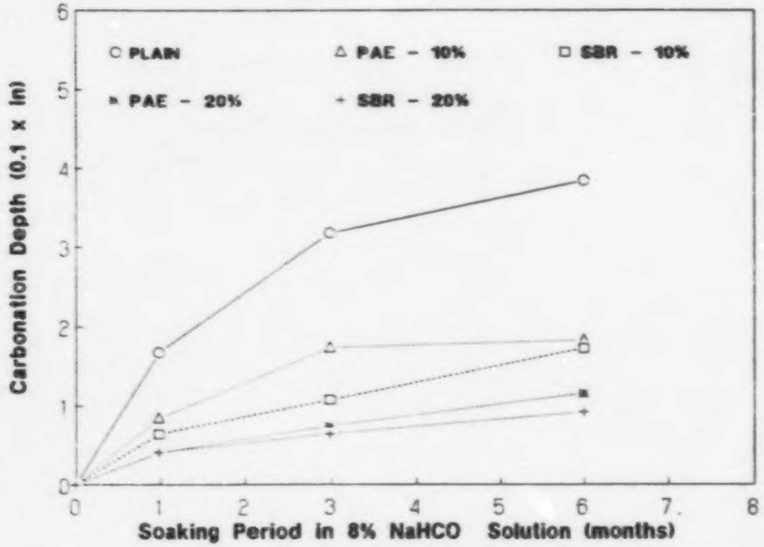

FIGURE 19 CARBONATION DEPTH Vs SOAKING PERIOD FOR UNMODIFIED Vs LATEX-MODIFIED CONCRETES (FROM REF.44)

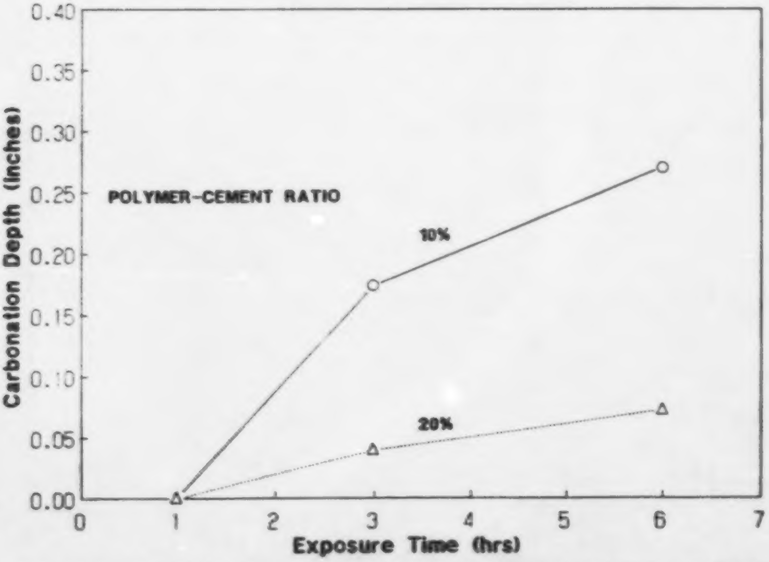

FIGURE 20 CARBONATION DEPTH OF SBR LATEX-MODIFIED CONCRETE Vs EXPOSURE TIME (FROM REF.44)

New Developments in Chemical Admixtures

By V.S.Ramachandran and N.P.Mailvaganam

Synopsis: Impressive advances have been made in the development of chemical admixtures in recent years and this is evidenced by the number of conferences held on this subject, the published literature and the number of patents appearing on admixtures annually. In this chapter major advances made in the field of admixtures, with special reference to their practical implications are reviewed. The discussions include: the synthesis of admixtures, the development of new admixtures, use of admixtures in new applications and their effect on concrete properties in the fresh and hardened state. The review discusses the following : 1) Development of lignosulfonate-based superplasticizers 2) Improvements of concrete properties with high dosages of super-plasticizers 3) Methods to control slump loss in superplasticized concrete 4) Novel techniques to synthesize superplasticizers 5) Development of high strength concrete 6) Innovative admixtures for winter concreting 7) Use of porous particles as air entraining agents 8) Polymer-based admixtures 9) Development of macrodefect-free cements 10) Activators for slags 11) Chemical Admixtures for reducing alkali-aggregate expansion 12) Admixtures for underwater concreting 13) Assessment of non-chloride admixtures 14) Stabilizers and activators for re-use of returned concrete.

Keywords: activators; accelerators; admixtures; air-entraining agents; alkali-aggregate expansion, high strength concrete, lignosulfonate, macrodefect-free cements; stabilizers; superplasticizers; underwater concreting

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1. INTRODUCTION

An admixture, according to the American Concrete Institute (ACI) document 116 R and ASTM C 125 is a material other than water, aggregate, hydraulic cement, and fiber reinforcement, used as an ingredient of concrete or mortar, and added to the batch immediately before or during its mixing. Chemical admixtures are not defined but include accelerators, retarders, water retarders, superplasticizers,etc. The ACI committee report entitled "Chemical Admixtures for Concrete" includes among others, air entraining admixtures, pumping aids, coloring admixture, alkali-aggregate expansion reducing admixtures and others. In this chapter chemical admixtures are defined as those exclusive of mineral admixtures.

Most of the chemical admixtures are used in small quantities and are water soluble or emulsified systems. A brief account of the important developments that have taken place in the last 5 years is emphasized. It is not meant to be an exhaustive survey but merely a review of the trends in the development of chemical admixtures. Patented literature has not been covered. Significant advances have particularly been made in the production and application of superplasticizers. Important developments are foreseen in such admixtures as lignosulfonates, accelerators, air-entraining admixtures, alkali-aggregate expansion reducing admixtures, admixtures for underwater concreting, polymer-modified and macrodefect free systems, activators and stabilizers for reuse of returned concrete, activators for slags,etc. The chapter lays emphasis on the improvements in water reduction, dispersion, workability, strength development, durability, etc in concrete containing the chemical admixtures. Aspects such as mechanisms, microstructural aspects, chemical reactions and new techniques of investigation are not discussed.

2. LIGNOSULFONATE-BASED SUPERPLASTICIZERS

In concrete applications lignosulfonate admixtures are used widely as normal water reducers. At a dosage of 0.05-0.1% they reduce the water requirement by 6-10%. At higher dosages excessive retardation of set and entrainment of air occurs and hence they cannot be used as high range water reducing admixtures. On the other hand, synthetic polymers based on the sulfonated melamine and naphthalene formaldehydes, while decreasing water requirements by about 20-30%, do not adversely affect the setting and air entraining properties of concrete. Because of the relatively lower price of lignosulfonates there has been continued interest in developing a superplasticizer based on lignosulfonates. Superplasticizers known as modified lignosulfonates contain some amount of the conventional superplasticizer. The properties obtained using them are comparable to those using only superplastizers. In Table 1 the properties of the concrete containing a modified lignosulfonate are compared with those with melamine and naphthalene-based superplasticizers (1). The results for two types of cement are similar.

It has been found that lignosulfonate-modified superplasticized flowing concrete with high slump and low cement contents may exhibit higher degree of deterioration due to freeze-thaw action and deicing chemicals, compared to the control concrete (2). In Table 2 are given the durability data for samples exposed to 300 cycles of freezing and thawing according to ASTM C 666, procedure A. The main type of deterioration is through the internal disruption of the specimens.

Attempts have been made to develop a synthetic superplasticizer based only on lignosulfonates. Rixom and Waddicor (3) prepared various types of lignosulfonates and studied their effect on concrete slump and water requirement. The significance of molecular weight, cationic species, degree of sulfonation and carbohydrate contents on initial set, workability and strength characteristics of concrete was studied. It was concluded that increase in molecular weight of lignosulfonate resulted in a more workable concrete, retardation of initial set and a slight reduction in compressive strength at 24h. Increase in sulfonation did not affect workability or entrained air but it reduced the setting time. It was also found that sodium lignosulfonate was a better plasticizer than calcium lignosulfonate. Generally increased amounts of sugars in lignosulfonate resulted in longer setting times.

A comparison of the effects of lignosulfonate, melamine sulfonate and naphthalene sulfonate admixtures was made. Sodium lignosulfonate was found to be a more effective plasticizer than others. The slump retention was also higher with sodium lignosulfonate. Table 3 illustrates the effect of three admixtures when used at double the normal dosage (3).

When lignosulfonate is used at a lower dosage, it decreases early strengths and prolongs the setting times. Incorporation of an accelerator decreases the setting time and increases the compressive strength. At normal dosage the setting time was 8 hrs and the 24 hr compressive strength, 10.2 MPa for concrete containing only lignosulfonate.

Ronneberg(4) has described the properties of concrete containing a lignosulfonate with low sugar contents. The plasticity was almost the same as with other superplasticizers. When it was used as a water reducer, the air content, and setting time were not different from those using other superplasticizers. When lignosulfonate was used to increase the slump of concrete the setting time was increased. At a dosage of 3L/m^3 (40% dry material) the retardation of setting times with sulfonated melamine, sulfonated naphthalene and lignosulfonate were 1.5, 3.5 and 7 h respectively. The 24 hr strength of reference concrete and that with lignosulfonate (1 L/m^3) was the same. However, at higher dosages of lignosulfonate the strength decreased. Same strengths were obtained when other superlasticizers were used at dosages 2-5L/m^3.

Recently Bialski and Nayak* attempted to synthesize a lignosulfonate for use as a superplasticizer. Spent liquor was subjected to heat treatment to reduce sugar content. The resultant product was subjected to ultrafiltration to remove low molecular weight materials including sugars. The molecular weight of the products was in the range 20,000 to 50,000. Several properties of the lignosulfonates and their effect on mortars were determined. The properties studied included surface tension, dispersion, and foaming characteristics of lignosulfonates. The properties studied on cement and mortar included viscosity, flow characteristics, setting times, water reduction and compressive strength. Of the many lignosulfonates studied five preparations imparted better mortar flow characteristics than those with a melamine-based superplasticizer. Some of the properties of mortars containing a lignosulfonate are compared with those using a melamine formaldehyde-based superplasticizer (Table 4).

Although a tailored lignosulfonate may qualify as a superplasticizer several problems associated with its use have to be solved. Many lignosulfonates may foam when used in mortars and increase the air entrainment. The setting times are generally increased. If the dosages of normal superplasticizers and those of lignosulfonates are kept low the compressive strengths of concrete will have about the same value. However, at higher dosages, concrete with lignosulfonate may show lower strengths. The use of defoamers and accelerators in lignosulfonate could offset some of the disadvantages mentioned above. Caution should be excercised in the selection of defoamers as some of them affect the aggregate-cement bond.

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*Bialski, A.M., and Nayak, K.V., "Superplasticizers from Lignosulfonate" (Private Communication)

3. DEVELOPMENTS IN THE SYNTHESIS OF SUPERPLASTICIZERS

Continued attempts are being made to futher improve the performance of superplasticizers. The effect of synthesizing conditions and molecular weight of superplasticizers on the behavior of cement, mortar and concrete have been reported.

A novel four step process has been developed for the synthesis of a sulfonated melamine formaldehyde. Over 20 superplasticizers were prepared by this process and the workability and mechanical properties of concrete incorporating them has been studied (5). The four steps included hydroxymethylation, sulfonation, low pH condensation and high pH re-arrangement. The results obtained with these superplasticizers were compared with those obtained with a standard concrete containing a melamine-based commercial superplasticizer.

Table 5 shows the relative water reducing capabilities of three superplasticizers at a slump of 50 mm (5).

A significant increase in water reduction occurs with the synthesized admixtures at lower dosages. It was also observed that the synthesized admixtures were effective in increasing the slump of concrete. For example, to obtain a slump of 200 mm, 1.3% commercial superplasticizer was required but the same value can be obtained with 1% of the newly synthesized superplasticizer. Slump retention, as well as strength was also greater with the synthesized admixture. At 28 days the strength of a neat mortar, that with 3% commercial superplasticizer and 3% synthesized admixture was 30, 28 and 34 MPa, respectively.

The rheological characteristics of cement or concrete are related to viscosity, zeta potential and also on the adsorption of admixtures on cement. The molecular weight of the admixture has an effect on rheology. Zeta potential values are indicative of the dispersive effects of admixtures. Andersen et al (6) studied the effect of the molecular weight of sulfonated polystyrenes (4000 - 70,000 g/mole) on their ability to disperse cement particles. The dispersion increased with the molecular weight and attained almost a constant value beyond the molecular weight of 40,000 g/mole.

Collepardi and coworkers (7,8) synthesized 20 naphthalene sulfonated formaldehyde samples of different degrees of polymerization. Both adsorption and zeta potential values of these admixtures increased with the molecular weight, up to a particular value. Strength of mortars increased with the degree of polymerization . A plot of the monomer content vs minislump of cement paste showed a steady decrease in values as the monomer content increased, the minislump values being 175 and 25 cm^2 at monomer contents of 10 and 100%, respectively.

As has already been discussed the molecular weight of the superplasticizer has a strong influence on the rheological properties of the cement systems. The cement composition, the alkali content and particle distribution are other factors that have to be considered. Ferrari and others (9) found that sulfonated naphthalene-based superplasticizers exerted the best fluidizing effect in the molecular weight range 6000-8000 (weight average) but at higher range the effect decreased. These cements were rich in C_3A. Cements that were rich in C_4AF did not show this drop at higher molecular weights.

4. HEAVILY DOSED SUPERPLASTICIZED CONCRETE SYSTEMS

Important benefits result in certain concrete systems by using higher than normal dosage of superplasticizers. In an investigation of the possibility of using high volume fly ashes in concrete, Langley and co-workers (10) succeeded in making durable fly ash concrete comprising 50-60% class F fly ash (of the total cementitious material) low unit water content equal to $w/c < 0.35$, and a superplasticizer content of up to 7.9 L/m^3. Although the early strength gain of control mixes containing no fly ash was higher than that with fly ash, a significant gain in strength for the fly ash concrete occurred from 7 to 28 days. For example, the mixture with $150 kg/m^3$ cement and $190 kg/m^3$ fly ash showed a strength of 25.8, 46.1 and 61.8 MPa at 7, 28 and 91 days and the reference concrete without fly ash had a strength of 38.7, 47.2 and 51.3 MPa, respectively. In spite of relatively low cement factor, the fly ash concrete showed desirable mechanical properties, satisfactory freeze thaw resistance and low permeability to chloride ions.

Rodway and Fedirko(11) also examined concrete containing up to 70% fly ash (high calcium) and superplasticizer (twice the normal dosage) and studied properties such as compressive strength, flexural strength, air void parameter, creep and permeability. The main drawback was delayed setting time. Otherwise the properties were adequate for structural applications.

Fly ashes are incorporated into concrete to counter the expansion caused by alkali aggregate reactions. Alasali and Malhotra(12) studied the effect of higher than normal additions of fly ash (50-60%) in concrete (by using higher than normal dosages of superplasticizer) and determined the inhibiting effect of this concrete to alkali aggregate expansion. For example, the reference concrete containing alkali and exposed to normal sodium hydroxide at 80°C for 275 days showed an expansion of 0.409% whereas that containing fly ash and alkali exhibited an expansion of only 0.083%.

Mine tailing dumps and some rock outcrops are susceptible to attack by penetrating rain and surface water. The leachates that result from this action may be potentially toxic and cause environmental and health problems. A new capping technology, using high volume fly ash shotcrete has been suggested (13). Several mixes were used in which cement contents ranged from 123 to 152 kg/m^3, fly ash contents between 193 and 328 kg/m^3, polypropylene addition from 4 to 5 kg/m^3 and superplasticizer dosage between 3.6 and 7.8 L/m^3. Properties such as compressive strength, flexural strength, flexural toughness, impact behavior, drying shrinkage, chloride permeability and freezing and thawing durability were determined. The economics and advantages of using this mix were discussed with reference to other methods.

The use of superplasticizer and excess air entraining agent permits production of high volume blast furnace slag. In the work of Nishibayashi et al(14), concrete containing 85% slag was used. Regardless of the percentage of the slag used the compressive strength of superplasticized concrete was almost equal to that of the control concrete. The initial setting time was unaffected but the final setting time increased as the amount of slag increased (14).

5. CONTROL OF SLUMP LOSS IN SUPERPLASTICIZED CONCRETE

Superplasticizers are known to increase the initial slump of concrete considerably. This increase however, is only transient because it is generally not maintained beyond about 30-60 min. One method is to add the superplasticizer at the point of discharge. There are some problems associated with this approach. Several other methods can be adopted to control slump loss. They include adding a higher than normal dosage of superplasticizer, redosing the superplasticizer at different intervals of time or by including some type of retarder in the formulation. Continued interest is evident from the research carried out on various methods of controlling slump loss. A few of the recent developments are discussed.

Precursor Dispersant

The effectiveness of superplasticizer is related to its concentration in the liquid phase. The concentration of superplasticizer decreases considerably, especially through strong adsorption in the early periods of hydration of cement. Fujiu et al(15) have developed a polymeric dispersant which by itself is insoluble but under the alkaline environment during cement hydration forms a product which has a superplasticizing effect. Thus by this time-delay mechanism the admixture is capable of maintaining the slump for a longer period. The polymer precursor has ester, acid chlorides, amides, and acid anhydrides as functional groups. The slump retention increases with the decrease in particle size as shown in Fig.1.

Reactive Polymer

A new slump retaining admixture has been reported. It is based on polymers. It consists of a naphthalene sulfonate based superplasticizer and a reactive polymer. Concrete with this admixture exhibits better slump retention characteristics than the reference concrete containing only the naphthalene-based superplasticizer. For example, at 90 min the slump values using the new formulation and that containing only naphthalene sulfonate formaldehyde are about 20 and 14 cm respectively (16).

Polycarboxylic Acids

Several types of polycarboxylic acid dispersants have been examined for their effect on slump increase and slump retention. Both the initial slump values and slump retention times were found to increase by the use of polycarboxylic acids at a level of 0.12% by weight of cement. The optimum molecular weight of the superplasticizer was found to be 10,000(17). In general, this admixture in comparison with the naphthalene-type superplasticizer, imparted lower bleeding, higher slump and slump retention but the concrete had higher air content and longer setting times.

Latex Modified Systems

Ramachandran and co-workers(18) investigated the effect of different dosages of various latexes on the slump of superplasticized concrete. The styrene-butadiene(P_1) latex was most effective in controlling the slump loss in a melamine formaldehyde-based(S_1) superplasticized concrete. The initial slump increased further when 2% latex was added to concrete containing 0.3% superplasticizer. Fig.2 compares the slump retention characteristics of concrete containing 0.3%S_1, 0.3%S_1+1% or 2% P_1. The concrete containing no P_1 has a steep rate of slump loss, attaining a value of about 5 cm at 4 h. By adding 1 and 2% P_1 to 0.3% S_1 the slump retention at 4 h is relatively higher than that without P_1, being about 8 cm and 14 cm, respectively.

6. ULTRA HIGH STRENGTH SILICA FUME CONCRETE

Normal weight structural concrete members have strengths between 23.5 and 35.2 MPa. Utilization of superplasticizers enables field placed concrete to attain strengths of 60 MPa (19). Ultra high strength concrete defined as that having compressive strengths from 68 to 124 MPa and a flexural strength of 14MPa may be fabricated at a water/cement of 0.22. In silica fume concrete, addition of superplasticizer decreases the water requirement, thus increasing the strength. The pozzolanic action of the highly reactive silica fume also contributes to concrete strength. Further enhancement of strength is achieved using a high cement factor and stronger aggregates. With 593 kg/m^3 ASTM Type I cement, 119 kg/m^3 silica fume (W/C=0.22, slump 11.9 cm) a strength of 111.4 MPa at 28 days and 124 MPa at 128 days may be achieved (20).

Mitsui and coworkers (21) have carried out a comprehensive investigation on the production of high strength silica fume concrete utilizing an admixture consisting of a sulfonated naphthalene formaldehyde and a copolymer with a functional sulfonic and carboxylic group. In Fig. 3 the strength development at 28 days for concrete made at different water/cement+silica fume and superplasticizer dosages 0.8% to 5% is given. It is obvious that the highest strength is attained in the presence of 20% silica fume. At a W/C+SF (W=water; C=Cement; SF=Silica Fume) of 0.25 a strength of 105 MPa is obtained. The percentage increase in strength with silica fume is very high at a W/C+F of 0.35. The influence of silica fume on the strength development is particularly significant when concrete is fabricated at moderate W/C (22). Fig. 4 shows the effect of W/C ratio on the 28 day compressive strength of concrete containing 0, 8 and 16% silica fume. In the W/C ratio range of 0.4 to 0.9, compressive strength is always much higher in concrete containing 16% silica fume. The influence of silica fume on strength development increases as the W/C ratio is decreased. However, at very low W/C ratio the effectiveness is low.

Several other reports confirm that superplasticized silica fume concrete develops very high strengths. Aitcin and Riad obtained a strength of 85.9 MPa at a W/C 0.21(23). Malhotra (24) found that concrete containing 3.89% naphthalene based-superplasticizer (42% solids) and containing 8% silica fume and fabricated at a W/C=0.25 developed a strength of 74.2 MPa and 89.9 MPa at 28 and 91 days, respectively. The mix containing 8% silica fume and 20% slag exhibited a strength of 95.8 MPa.

Strengths of about 100 MPa were obtained by Collepardi et al(25) in concrete (slump of 20 cm) containing 10-20% silica fume, 2.4% superplasticizer and 400 kg/m of cement . Generally, silica fume concrete develops strengths at a low rate in the early periods of curing. High early strengths may be obtained using a rapid hardening cement in combination with a superplasticizer. Concrete containing 8% silica fume, a rapid hardening cement and superplasticizer may develop a strength of 34 MPa in 1 day (22).

7. ANTIFREEZING ADMIXTURES

Accelerating admixtures such as calcium chloride are used in cold weather concreting. A significant increase in the early strength development at normal and low temperature enables reduction in the curing and protection periods necessary to achieve specific strengths in concrete. Calcium chloride cannot be considered an antifreezing admixture. For example, at a dosage of 2% and a water:cement ratio of

0.50 the freezing point of water is lowered by about 1.4°C which is negligible from practical considerations.

Antifreezing admixtures are capable of depressing the freezing point of water in concrete considerably so that their use is of practical significance. Antifreezing admixtures are used in the USSR at temperatures as low as -30°C (1). These admixtures allow earlier stripping and use of forms, earlier completion of construction projects and occupancy. They are particularly useful in the fabrication of precast elements, for patching and repairs, foundation work, and slabs on grade for garages. Improved cohesiveness, plasticity, minimization of cold joints and sand streaking are some of the advantages.

The mode of action of the antifreezing admixture is a) to lower the freezing point of water in concrete and act as either a weak accelerator or retarder of setting and hardening. The admixtures that are used for this purpose include sodium nitrite, sodium chloride, weak electrolytes and organic compounds such as high molecular weight alcohols and carbamide, or b) to accelerate significantly the setting and hardening with good antifreezing action. In this group belong chemicals such as calcium chloride, sodium chloride, sodium nitrite, calcium nitrite-calcium nitrate and urea.

The Russian literature contains a significant amount of information on antifreezing admixtures (1). Only recently has some interest been evident from other sources. The RILEM committee recognizes antifreezing admixtures for cold weather concreting (26). It also includes cautions to be taken in the use of these admixtures. Generally, larger dosages of antifreezing admixtures are used compared to chemical admixtures. For example, with 8% sodium nitrite 50% of water will still be in the liquid state in concrete at a temperature of -15°C. In Table 6 the strengths (as a percentage of designed strength) are given for concrete cured at -5°C and containing sodium nitrite(SN), calcium nitrite-nitrate-chloride (CNNC), and calcium chloride-calcium nitrate (CC+SC). The binary admixture addition at lower dosages imparts higher strengths than when used singly.

In Finland premixed dry mix mortars containing antifreezing admixtures are marketed. They can be used even at -15°C and are specially applicable for joints and repair purposes. (27). Fig.5 shows the strength development in mortars with this product at -5, -15 and 20°C. Even higher strengths are achieved at -15°C beyond 28 days of curing.

It has also been shown that with the antifreezing admixtures the strength of concrete with ordinary cement may exceed that obtained with the rapid hardening cement, containing no admixture. Table 7 compares the results obtained with the two types of cement at -10°C. Best results (28) are obtained using a mixture of potassium carbonate and a retarder. Brook et al (29) have described an admixture called ATA, a non-chloride admixture that is non-corrosive, water reducing and set accelerating. With this admixture (10% solids) ice formation at -6.7°C can be prevented. In another publication Scanlon and Ryan (29A) have provided data on a new freeze-protection admixture. This admixture accelerates set time and accelerates strength development at sub-zero temperatures as shown in Table 8.

A non-freezing admixture has also been reported in the Japanese literature(30). Concrete containing this admixture when cured at -10°C showed strengths of 14, 75, 165 and 213Kg/mm^2 at 3, 7, 14 and 28 days respectively and the corresponding values for untreated concrete were 2, 3, 4 and 6Kg/mm^2. Continued interest in antifreezing admixtures is evident from other recent publications on potash-lignosulfonate-sodium tetraborate (31), potassium carbonate-lignosulfonate (32), nitrite-nitrate-alkali (33) and polyamine-lime-alumina cement-potash (34).

Much more information has to be collected before these admixtures can be used widely. Antifreezing admixtures are expensive especially as they are used in higher dosages. The hardening rate is sometimes too slow, the accelerating rates may be affected by plasticizers, and compatibility with other normal admixtures as well as their effect on long term durability has to be established for each mix and curing regime.

8. PARTICULATE ADMIXTURES AS SUBSTITUTES FOR AIR-ENTRAINING AGENTS

The general method of counteracting freezing and thawing attack on concrete is to use air-entraining admixtures. Factors such as variability in the material, mixing, placing methods, and temperature may make it difficult to adjust the required amount of air with the right bubble size and spacing. It is also not easy to adjust the air in blended cements, superplasticized concrete, and high strength concretes. Whiting and Stark (35) have listed 23 factors that affect air entrainment.

The problems associated with the use of air entraining agents are largely circumvented by adding preformed bubble reservoirs in the form of porous particles. In one method hollow plastic microspheres with diameters between 10 and 60 µm are added to concrete (36). The voids in the particle are smaller than those in the air entrained concrete (10-3000 µm). Addition of 1% by weight of cement of these microspheres to concrete corresponds to 0.7% by volume of concrete. The spacing factor equivalent using these spheres is 0.07 mm, well below the permissible maximum. Sommer (36) compared the losses in weight of concrete (due to freezing-deicing salt attack) containing 5% entrained air with that containing 0.3, 0.6, 0.9, 1.2 and 1.5% hollow microspheres. Adequate frost resistance was attained using 0.9% of the microspheres. No further advantage was observed using higher proportions. Thus 1% hollow sphere was found to be as effective as 5% entrained air. The workability as a measure of consistency was increased by the microsphere addition to the same extent as that provided by 5% air. The 28 day strength was higher with microspheres, being 40MPa and 38MPa with 1% hollow spheres and 5% air respectively.

Vanhanen(37) confirmed some of the findings of Sommer (36). Using 0.3, 0.6, 0.9 and 1.2% microspheres by weight of cement, concretes were made. The results of compressive strength and frost resistance of these mixes were compared with those of reference concretes (with and without air entrainment). The frost resistance was improved considerably in concrete containing microspheres. A minimum amount of 0.6% microsphere was needed for good frost resistance. It was also concluded that the compressive strength (with respect to the plain non-air entrained concrete) of air entrained concrete was less than that containing microspheres.

Recently Litvan(38) demonstrated that porous particles made with fired clay bricks, diatomaceous earth, vermiculite, pumice and perlite can be added to concrete to increase its frost resistance. These results show that concretes containing porous particles are generally more durable than the plain concrete.

In Table 9 are given the particulate admixture used, its concentration in concrete, and the compressive strength. Two concentrations were added to concrete to assess their effect on freezing and thawing durability. In general, the addition of particles (with the exception of brick) lowered compressive strength. At lower particle concentration the reduction of compressive strength is 7.7%, 6.1% and 19.5% for vermiculite, pumice I and perlite, respectively. These differences will be lower if comparison is made with respect to the air-entrained concrete. The lower strengths should not be a problem for applications related to sidewalks, roadways and other horizontal surfaces where better durability promoted by these particulates is expected to compensate for a slight decrease in compressive strength.

The expansion and dynamic modulii of elasticity of concrete specimens exposed to different number of cycles of freezing and thawing cycles are shown in Figs 6A and 6B(39). Results indicate that concrete containing higher concentrations of particulates (Table 9) showed high freezing and thawing resistance even after exposure to 350-400 cycles. The only exception was concrete with pumice II. The plain reference concrete (containing no air-entrainment) failed after 60 cycles at a w/c = 0.60 and that fabricated at a w/c of 0.56 failed after 300 cycles, using ASTM C-666 precedure B.

9. POLYMER-MODIFIED MORTAR AND CONCRETE

Polymer-modified mortars and concretes are used as construction materials for finishing and repair purposes. They are widely used for bridge deck overlays and patching, and also as adhesives, anticorrosive liners, decorative coatings and integral waterproofers. It is estimated that annually 1.2 million sq metres of bridge decks are overlayed with polymer-modified concrete (40). The polymers are usually added to the mixing water just as other chemical admixtures, at a dosage of 5-20% by weight of cement. There is evidence that interactions occur between polymers and hydrating cement (41-42). Polymers influence the setting, bleeding, workability, hydration and mechanical properties of cement. There is continued interest to develop polymer-based admixtures for various new applications in concrete.

The polymers consist of latices, liquid resins and those that are water soluble. The latices are more widely used than others. The major types of latices (dispersed suspensions of solid polymer microparticles in water) comprise styrene butadiene (SBR), ethylene-vinyl acetate (EVA), polyacrylic ester (PAE) and epoxy resin (EP).

Ohama, et al (43) have developed a highly durable concrete using polymer dispersions containing 0.5% alkyl silane. The polymer dispersion consisted of SBR latex, EVA emulsion and PAE emulsion at polymer/ cement ratios from 5 to 20 %. Properties such as strength, chloride ion penetration, carbonation and drying shrinkage were determined. All polymer-modified concretes showed superior properties to those of the unmodified concrete. In the EVA and SBR concrete,the chloride penetration depth was significantly lower than that in PAE-modified concrete. The penetration depth in the polymer-modified concrete was almost half of that in the unmodified concrete (Fig.7).

Polymer-modified mortars may be used for repairs but they harden slowly, so that their application on substrates with flowing or leaking water is not very effective. An ultrarapid hardening shotcrete (URH) has been developed which contains magnesium acrylate in combination with sodium persulfate and sodium ascorbate (44). The URH was reported to stop water leaking at a water pressure up to 0.01 MPa. The adhesion in tension of URH shotcrete was 2MPa, which was about 10 times that of the un-modified mortar. The flexural strength and the freezing and thawing resistance of URH shotcrete were also higher than those of the un-modified mortar. Shotcretes containing silica fume and latex are known to have a better rebound and bond strength characteristics.

Significant increases in compressive strengths in concrete are possible utilizing polymer admixtures. Adequate strength increases are achieved using an epoxy resin, in combination with a superplasticizer or an accelerator. Fig.8 compares the rate of strength development in a control concrete, epoxy-modified concrete and epoxy-modified concrete with a superplasticizer (45). High strengths were obtained with a mixture of epoxy and the superplasticizer;the 7 day strength was 60% higher than that of the control concrete and 23% higher than that of the epoxy-modified concrete without a superplasticizer. Use of a chloride-free accelerator also produced strength increases of 10-20% at 28 days. Generally, incorporation of polymers is known to increase the flexural strength of concrete. Significant increases in flexural strengths were achieved with polymethyl methacrylate copolymer (0-20%) and an antifoaming agent (46). A cement containing these admixtures yielded a flexural strength of 30 MPa, Young's modulus of 15 GPa and the work of fracture of 555 J m^{-2}. The corresponding values for the cement paste containing no admixtures were 12 MPa, 8 GPa and 125 Jm^{-2} respectively. The possibility of increasing the ductility of concrete by incorporating a mixture of natural rubber latex and a superplasticizer has been examined (47). The addition of superplasticizer prevented premature coagulation and increased the flow of cement mortar. With dry rubber contents up to 2% the compressive strength and tensile strengths remained unaltered but the strain capacity increased significantly. This was reflected in the marked improvement of the ductile behavior of latex concrete. It was found that up to 2% rubber content, there was no difference in the compressive (Fig. 9) and tensile strengths of latex and normal concretes ; however, there was a marked increase in the strain capacity in the latex concrete. Thus there is an impressive increase in the ductile behavior of polymer-modified concrete. Beyond 2% rubber addition, although strength decreases by about 15-30%, the ductile behavior remains very pronounced.

Extension of workability, superior bonding strength and increased ratio of bonding: compressive strength have been demonstrated in a polymer-modified mortar containing an acrylic latex and a superlasticizer (48). For example, the reference mortar containing 0% polymer had a bonding : compressive strength ratio of 0.2, whereas the mortar containing 15% polymer, 15%polymer+ superplasticizer and 15% polymer + silica fume developed ratios of 0.38. 0.31 and 0.37 respectively.

10. MACRODEFECT-FREE CEMENTS

The compressive strength of portland cement ranges from 27.5 to 55MPa, and increases to about 138MPa with silica fume. The macro defect-free cement having high strength was reported in the early 80's by the Imperial Chemical Industries (49-50). Strength surpassing 276MPa was obtained with MDF cements. In the MDF cement the constitutents include the cement and a polymer such as PVA, polyacrylamide or hydroxypropyl methyl cellulose. A low water:cement ratio of about 0.10 to 0.15 is used. The cement acts as a reactive filler and it is only partially hydrated. Since the hydrated cement may act as a cross-linking agent with the polymer, the polymer should be considered as an admixture, although it is used in large amounts. Special processing techniques are necessary for fabricating MDF cements. Roll milling and mixing at high shear rates are involved. Full strength development is achieved by curing at about 80-120°C for about 24 h.

The cement containing PVA and high alumina cement is sensitive to moisture, the flexural strength decreasing with exposure to moisture. Improvements are achieved by insolubilizing PVA matrix with cross linking or coupling agents. Improved strengths are obtained by inclusion of an epoxy functional group and boric acid or organic chromium complexes as cross-linking agents (51). A flexural strength as high as 250-300 MPa has been obtained with epoxy functional silane coupling agent(Fig 10)(51).

11. ACTIVATOR ADMIXTURES FOR SLAGS

The activation of granulated blast furnace slag in slag concrete is generally based on the alkalis released from the hydration of portland cement. Use of slag as a cement without portland cement has advantages. Because the strength development of slag by itself occurs at a slower rate than that of portland cement, much attention has been directed to a study of the activation of slag containing no cement. Activators are of three types. viz, caustic alkalis (Na, K or Li hydroxide) non-silicate salts of weak acids (R_2CO_3, R_2SO_3, R_2S or RF; R=Na, K or Li) and silicate salts of type $(R_2O)(n).SiO_2$. All three types are used for basic slags but for neutral or acid slags non-silicate salts of weak acids are not recommended.

The types of activators used include sodium hydroxide, sodium carbonate, F-activator produced in Finland (contains NaOH, Na_2CO_3 and lignosulfonate) sodium silicate, sodium sulfate, potassium sulfate, lime and their combinations. For better workability and low water requirements admixtures such as lignosulfonates, gluconates and superplasticizers have also been added with activators. The rate of hydration of slags is enhanced using finely ground slag and higher temperatures. The degree to which the slag is activated depends not only on the type and amount of the activator, temperature and fineness but also on the chemical composition and glass content of slag. In view of this the best activator suggested for a particular slag may not be applicable for another slag. Some of the relevant reports published recently are discussed below.

With F concrete, (Finnish formulated concrete) using alkali activators, Byfors et al(52) concluded that best strengths are obtained with 2% NaOH + 2% Na_2CO_3. Talling(53) achieved optimum strengths in slag-mortars with sodium silicate activators containing 3-6% Na_2O. Gjorv(54) studied the alkali activator in combination with the Finnish formulation 'F on physical and mechanical properties of slags of different Blaine surface areas, and at different temperatures. With a Blaine surface area of $640 m^2/kg$, strength (at 24 h) attained a value of 22.6 MPa. A review of alkali-activators was published by Talling and Brandstetr (55). The strength development in a normally cured slag mortar containing sodium silicate of $M_S = 2$ (SiO_2/Na_2O ratio) and made at a water/slag of 0.32-0.35, was reported to be 10-20 MPa at 8 h, 30-42 MPa at 1 day, and 55-85 MPa at 28 days. Further increase in strength occurred at higher temperatures. It was also found that the mix was sticky when sodium silicate was used; addition of limestone powder, fly ash, silica fume, granite, etc improved the workability. In another study high strengths were obtained using 7% NaOH for a slag of Blaine 1000 m^2/kg. Also in an alkali-activated fly ash mixtures with 7% NaOH the strengths at 3, 7 and 28 days were reported to be 12-13, 14-15 and 20 MPa, respectively, and the corresponding values for slags were 16, 18 and 36 MPa (56). Activation of phosphorus slag is also possible with cement clinker, Na_2SO_4 and sodium silicate (57). Douglas and Brandstetr(58) have found that activated slag mortars with dilute solution of sodium silicate, 2% lime and 1% Na_2SO_4 gave compressive strengths comparable to ASTM C-109, portland cement mortars. For example, the strengths of mortars at a water/binder ratio of 0.48 were 13.4, 29.4 and 35.7 MPa at 1, 7 and 28 days respectively. Investigating slags with surface areas between 147 and 654 m^2/kg and seven NaOH activator concentrations from 1.6 to 30% and water/slag ratio of 0.3, 0.4, 0.6 and 0.7 and a superplasticizer, Isozaki et al(59) obtained effective strengths with 2-3% NaOH. Accelerated heat effects were also observed with NaOH or Na_2CO_3-NaOH mixtures in slags.

In a comprehensive investigation, using pastes and mortars, Andersson et al(60) evaluated the activation of slags using NaOH, Na_2CO_3 and water glass of general composition, Na_2O (n). SiO_2, where n varied between 0.9 and 3.35. Properties such as setting time, compressive strength, flexural strength, porosity, dynamic modulus of elasticity and shrinkage were studied. In general, setting time decreased in activated slag mortars. In several cases activated mortars produced higher strengths than the reference mortars. The strength development in reference mortar and that containing 8% sodium silicate or 11% NaOH were comparable. The mortars were all cured at 20°C. The water/binder was 0.43 for the activated mortars and 0.50 for the reference mortar. Only slag containing NaOH had sufficient strength for it to be stripped from the form at an early age. However, at 7 days and after the sodium silicate-activated slag had higher strengths (Fig 11)(60). At 28 days sodium silicate-activated slag had strength in excess of 50 MPa, whereas the reference concrete showed a strength of about 40 MPa. Shrinkage values (at 231 days) of activated mortars were generally higher, the values being in the range 0.093 - 0.22% compared to 0.068 for the reference mortar.

The products formed in the activated slags are reported to be low-basic calcium silicates, alkali-alumino silicate hydrates, calcium-rich magnesium silicate hydrate, $2CaO.Al_2O_3.2SiO_2.8H_2O$, hydrogarnet, silicic acid, zeolite type compounds, etc. At high temperatures crystalline compounds abound. No $Ca(OH_2)$ is detected.

Although there are several advantages of using activated slags there are some problems associated with their use viz, quick setting, sticky consistency, efflorescence effects, increased alkali-aggregate expansion, higher carbonation, irritation to the skin by the use of some activators, and higher costs.

Much more work needs to be done on activators for resolving some of the apparently contradictory results reported in the literature.

12. ALKALI-AGGREGATE EXPANSION REDUCING ADMIXTURES

One of the well known methods of reducing the expansion of concrete caused by alkali-aggregate reaction is to incorporate pozzolans in the mix. The possibility of using chemicals to reduce expansion was first suggested in the early 50's. The materials that have been studied include salts of lithium, barium and sodium, proteinaceous air-entraining agents, aluminum powder, copper sulfate, water reducing and set retarding agents. The expansion of concrete containing reactive limestone aggregate could be reduced by adding lithium and ferric chloride (1). Any chemical that is used should not only reduce expansion but also not adversely affect other concrete properties such as setting times, mechanical characteristics and durability. Because of limited laboratory data no recommended practices have been formulated pertaining to the use of these admixtures (61).

There has been a resurgence of interest in the development of a chemical to reduce the expansion caused by alkali-aggregate reaction. Ohama et al (62) investigated the effect of sodium silicofluoride, alkyl alkoxy silane, lithium carbonate, lithium fluoride, styrene-butadiene rubber latex and lithium hydroxide on the expansion of mortar containing cement with 2% equivalent Na_2O. Compressive strength measurements were also carried out. In Fig.12 the expansion results are plotted. The reference mortar without the chemical shows an expansion of 48×10^{-4}. The expansion is only 26×10^{-4} with 0.5% sodium silicofluoride. This was attributed to the formation of an insoluble compound with strong siloxane linkages. The alkyl alkoxy silane is also very effective in reducing the expansion of the reference mortar by 95%. Lithium compounds reduce expansion, but to a lesser extent than other compounds. The results of the relative compressive strengths indicate that the mortar with sodium silicofluoride shows the highest strength, being about 50% higher than that of the reference. The addition of other admixtures did not result in changes in strength.

The work by Sakaguchi et al(63) on the effect of lithium carbonate, lithium nitrite and lithium hydroxide showed that all these compounds effectively decreased the expansion due to alkali-silica reactions. Fog example, at a $LiOH.H_2O$ dosage of 0.65% the expansion reduced from about 0.25% to 0.1%. The effect of lithium compounds is attributed to the formation of lithium silicate that dissolves at the surface of the aggregate without causing swelling (63).

A method of treating the reactive aggregate before making concrete or treating the hardened concrete with calcium phosphate has been proposed as a method of countering the alkali-silica expansion. The expansion of a reference concrete containing a non-reactive aggregate was compared with that containing a reactive aggregate treated with monocalcium phosphate monohydrate. The aggregate treated with phosphate even for a minute had almost no expansion at 28 days, compared to an expansion of about 1% in the reference concrete (64). This method was less effective for concrete containing reactive carbonate aggregates. Phosphate addition may interfere with the dissolution of silica gel and the formation of gel. It is also possible that phosphate reduces the osmotic potential and the swelling pressure in the gel.

13. ANTIWASHOUT ADMIXTURES

Dewatering of hydraulic structures for repair is difficult and expensive. Recent advances in cohesion-inducing admixtures have allowed placement of concrete under water without the use of conventional tremies. The concrete is cohesive enough to allow limited exposure to the water, yet has good mobility to move underwater with little loss of cement. Such cohesion inducing admixtures are referred to as antiwashout admixtures (AWA's).

Most AWA's are composed of water soluble cellulose ethers or water soluble acrylic-type polymers as the main component (65).The action of admixtures is to increase the viscosity of water in the mix. This results in an increased thixotropy of the concrete and an improved resistance to segregation. Dosage of the admixture ranges from 1-1.5% by weight of the water in the mix and it is frequently used in combination with a superplasticiser (66) . The magnitude of the effect produced is dependent on the admixture dosage and the molecular weight of the main component. It is usually discharged into the mixer at the same time as the other materials.

The following effects are produced when the admixture is used in concrete:

- Cohesiveness, mobility, and self-levelling characteristics are increased.
- Improved resistance to segregation. When the concrete is allowed to free-fall through water, there is hardly any segregation.
- The pH value and turbidity of the water is decreased with increased dosage.
- Concretes containing cellulose ether type AWA will have an extended set time. Those containing acrylic-type do not extend the set time except when combined with superplasticizers.
- Because of the high water retentivity these admixtures induce in concrete, there is hardly any bleeding.
- The ratio of compressive strength of specimens in water to the strength of specimens in air at 28 days increases with dosage.

Typical applications of AWA is in the production of nondispersible colloidal underwater concrete. Minimum w/c range from 0.36 - 0.40. Cement and fine aggregate contents are usually higher than corresponding mixes placed on land. Silica fume may be used in conjunction with a superplasticizer or conventional water reducers to reduce segregation. Often it is difficult to adjust the mixture proportions to achieve desired design parameters for all properties of concrete. Consequently the properties of colloidal underwater concrete are controlled by the addition of three chemical admixtures. The key to a non-dispersible concrete with self levelling characteristics is the successful optimization of the AWA with the superplasticizer used to increase the slump.

Laboratory evaluation of non-dispersible concretes has been carried out to study the effect of various combinations of water reducers and supersplasticizers(65-68). Conventional lignosulfonate and hydroxy carboxylic acid water reducers and naphthalene formaldehydesulfonate (NFS) and melamine formaldehyde sulfonate (MFS)-based superplasticizers were used in these studies. The Tatterstal two point workability test and wash out test were used to evaluate the mix characteristics. Best results were obtained with lignosulfonates and MFS. The NFS superplasticizers produced erratic results in flowability and cohesion(66).

The properties studied led to the following conclusions . Spread and air content is directly related to mobility; turbidity and pH indicate the seepage of the hydrating cement in the area of concrete placement. Flowability directly affects the steel reinforcement to concrete bond. Bleeding affects permeability and durability while setting time is important for placement. Compressive strength and ratio of strength (water to air) are important to design underwater concrete structures.

Field tests have also been carried out on a commercially available AWA(67) The trials evaluated bonding capability, pumpability and flowability around reinforcement. The quality of the test pieces recovered and strengths of cores show AWA concrete to be suitable material for in situ structural concrete construction at considerable water depths (67).

Other work has centered on the use of auxillary agents to reduce the amount of polymer used in the concrete for cost reduction purposes (69). This agent reportedly reacts with the polymer in the AWA to increase the apparent molecular weight of the polymer thereby improving its cohesion inducing properties. Table 10 shows the results obtained from a study of concrete mixes containing the auxillary agent glutaraldehyde combined with a polyacrylamide AWA. It can be seen that the auxillary agent increases resistance to cement washout (as measured by transmittance) permitting a reduction of the polymer by 0.8% by weight, without any adverse effect on strength.

Non-dispersible concrete can be poured into a water filled form without a tremie pipe to produce dense structural repairs. This type of material has particular advantages over conventional concrete both in terms of the quality of the repair produced and the reduction in placement cost associated with plant and diver manpower requirements. New valves and control devices under development in Europe and Japan used in conjunction with AWA will likely advance the field on underwater concrete.

14. NON-CHLORIDE ACCELERATING ADMIXTURES

A large amount of data is available on the use of calcium chloride as an accelerator in concrete (70). The presence of chloride ions in concrete however, tends to increase the potential for corrosion of the embedded steel and hence there has been continued interest to develop an alternative to calcium chloride that accelerates cement hydration but does not promote corrosion of steel. Soluble salts such as bromides, fluorides, carbonates, thiocyanates, nitrates, nitrites, thiosulfates, silicates, aluminates and alkali hydroxides are known to accelerate the setting of cement. Organic compounds such as triethanolamine and formates also act as accelerators. Commercial non-chloride accelerators contain a combination of admixtures and their short and long term effects on concrete are yet to be completely assessed.

Rear and Chin(71) tested 3 commercially available non-chloride admixtures for their effect on early strength development of concretes with five ASTM Type I and Type I/II cements, two ASTM class F fly ashes and two ASTM C fly ashes. The testing was carried out at 22°C and 10°C. The three accelerators studied were Ca-nitrite, Ca-nitrate and Na-thiocyanate. At 22°C with the Ca-nitrite and Na-thiocyanate admixtures the one day strength of concrete increased by 128-144% and the 7 day strength by 109-131%. Not much improvement resulted with Ca-nitrate. At 10°C however, all the admixtures promoted higher strengths wth respect to the reference concrete at 1 day and 7 days. In class C fly ash concrete containing three types of cements, the Ca-nitrite based admixture promoted strength in two of them. In class F fly ash concrete two cements showed higher strengths at 1 day and all cements showed higher strengths at 7 days with respect to the reference concrete containing no calcium nitrite. At 28 days generally calcium nitrite enhances the strengths of concrete containing all the 3 cements.

Brook et al(72) have described the effect of a new non-chloride accelerating admixture on concrete containing a fly ash(called class CF) in which the CaO content was 5-15%. The setting, strength and other characteristics of the concrete were investigated at 21°C and 10°C. The accelerator shortened the setting times by 80-180 min and increased the compressive strengths at 3, 7 and 28 days. For example, the 3-day strength of the plain concrete was about 24MPa and that containing an accelerator dosage of 2.8L/100kg of cement, had a strength of about 31MPa. The corresponding values at 28 days were 41.4 and 44.8MPa. Although the flexural strength at 3 days was higher for concrete with the accelerator, the 7 and 28 day strengths were only equal to those of the plain concrete. There was no deleterious effect of the accelerator on shrinkage and air void characteristics. The accelerator had also beneficial effects on concrete made with class F and C fly ashes.

Many other non-chlorides have been studied for their action on the hydration and strength development in cements, mortars and concrete. Kuroda et al(73) found acceleration of setting and strength development using a multicomponent Calcium nitrite-Calcium rhodonate-triethanolamine admixture in concrete. Triethanolamine was found to be an accelerator for the slag cement (74). A water soluble organic material of the carboxylic acid-type was investigated for its effect on cement, mortar and concrete by Popovics (75). At a dosage of 2% the 3 and 7 day-strengths increased in comparison with the reference mortars. Sodium molybdate as well molybdenum oxides have also been found to be accelerators for cement but they are not as efficient as calcium chloride (76,77).

15. ADMIXTURES FOR RE-USE OF RETURNED CONCRETE

Every year millions of m^3 of the ready mixed concrete produced are returned for disposal in dump sites. There are now increasing environmental concerns and restrictions regulating the disposal of returned plastic concrete. The Environment Protection Agency classifies both returned plastic concrete and truck wash water as hazardous waste. Consequently, the conventional methods of disposal have become redundant, and disposal of plastic concrete is now a major problem for the ready-mixed concrete industry (78).

In an effort to assist concrete producers, to comply with new environmental regulations, admixture manufacturers have developed a cost effective alternative to disposal. The technique keeps concrete plastic for longer periods making possible its storage and later use. The system uses two non-chloride admixtures (stabilizer and activator), which alternately suspend and reactivate cement hydration. In this system a stabilizer coats cement particles to suspend hydration and interacts with hydration products that are in solution. Depending on the dosage, the mix can be maintained in an de-activated state for hours or days. An activator is then used to remove the coating, allowing hydration to proceed. The admixture system may be used the same day during long hauls, long waiting periods as a consequence of pump, truck or equipment breakdown, thus permitting the reuse of aged plastic concrete. Overnight/weekend stabilization of both returned plastic concrete and truck wash water for reuse as part of the mix batched the next day, is another important application.

Specifically, concrete returned to a ready-mixed concrete plant is stabilized (cement hydration stopped) in the ready mix truck, or collection containers. The treated concrete remains in the transit mix truck or designated containers until needed. In most cases the reclaimed concrete is stabilized for 1-3 days. When ready for use, the activator admixture is added, and the stabilized/activated concrete is blended with fresh concrete. The system generally blends 15-35% reclaimed concrete with 65-85% fresh concrete. It is generally used by authorized ready-mixed concrete suppliers fully trained (by the admixture manufacturers) to determine quantities of stabilizer and activator for each mix. A team of researchers visits each plant and develops either a computer program or a set of charts to optimize the dosage rates for the customer according to the variables mentioned below.

The stabilizer is usually used at a dosage ranging from 0.325-8.47 L/100kg of cementitious materials. Dosage is influenced by a number of factors including, (a) mix proportioning, (b) elapsed time between batching and stabilizing, (c) average ambient temperatures, (d) quantity of reclaimed concrete and (e) desired time of stabilization. The activator dosage range is 0.65-0.98 L/100kg of cementitious material and is also governed by most of the factors influencing the stabilizer dosage. Unlike conventional retarders, the stabilizer can be used at high dosages without the attendent adverse effects such as flash set and poor strength development characteristics resulting from the use of normal retarders (79).

The stabilizer may consist of carboxylic acids and phosphorus-containing organic acid salts. The mechanism of action of the stabilizer admixture is thought to be related to the inhibition of CSH and CH nucleation. It is claimed that the nucleation process is controlled more comprehensively than that obtained with conventional retarders (80). Cement hydration is arrested by the admixture acting on all phases of cement hydration including the C_3A fraction. The claim is supported by conduction calorimetric data and scanning electron microscopic examination of the surfaces of stabilized and activated alites. The stabilizer is a surface active agent designed to prevent surface nucleation of calcium ion rich hydrates. When nuclei have already formed, the stabilizer retards their growth and alters the external morphology of subsequently formed hydrates. The stabilizers' ability to stop formation of primary CSH hydrates and its moderate slowing of C_3A suggests that in Portland Cement, the stabilizer prevents epitactic growth of primary CSH on C_3S, while only slightly slowing C_3A hydrates from precipitating out of solution. Although the action on hydrate nucleation cannot be directly attributed to these admixtures, there are data which indirectly illustrate these effects. For example, the alteration of ettringite morphology on alite, and the more open texture of ettringite on C_3A suggests that these admixtures strongly influence how the hydrates are formed (80).

Manufacturers claim that the admixture has been tested for its effect on setting, air entrainment, freezing-thawing resistance, mixes containing fly ash and slag, flexural and compressive strengths, length change, susceptibility to cracking and corrosion and creep. The reported field and lab data show that no adverse effects are produced by the admixtures on these properties (79). Under same experimental conditions the same cement produces identical hydration products both with treated and control cement pastes as evidenced by X-ray diffraction, DTA, TGA and non-evaporable water determination. The physical structure as revealed by water vapor sorption and mercury porosimetry was also the same for both pastes (81).

16: CONCLUSIONS

There has been a continuous activity in the development of admixtures as evidenced by published literature and patents. In the year 1985-1989 nearly 600 patents were taken on concrete admixtures and the enormous interest in their development is evident from an impressive number of 242 patents reported in 1989.

In construction, there will be an increasing demand for the development of concrete with high performance characterisrics. Such concrete should be able to perform adequately against hostile environments when used in ocean sea floor tunnels, off-shore structures, sewage pipes, and in structures exposed to hazard chemicals and radioactive materials. Special types of concrete have also to be developed for repairs, and for infrastructural applications. Utilization of waste materials in concrete and solidification of hazardous materials in concrete will also require special mixes. For all such applications utilization of novel admixture combinations will have to be developed. Multifunctional admixtures for various purposes, in which a combination of admixtures would be necessary will be developed in the coming years. Greater attention will be directed to the production of more economical admixtures utilizing new techniques and raw materials derived from chemical wastes. The production of some of the admixtures presently used, such as lignosulfonates may pose environmental concerns; hence new methods will have to be devised to synthesize them or alternative admixtures have to be developed. Although non-chloride admixtures are being used presently they are not as effficient as calcium chloride. Therefore there is a great need to develop an admixture as efficient as calcium chloride. Efforts will continue in the field of advanced cement systems to produce materials of very high strengths and for such systems novel admixtures will have to be developed. The antifreezing admixtures are not popularly used in North America. Some work has been carried out on these admixtures and it can be foreseen that more attention will be directed towards the production of these admixtures in the future. Also more work will be carried out to develop economical admixture combinations for the re-use of returned concrete in the ready-mixed concrete plants. Although research has shown that some chemicals retard the alkali-aggregate expansion reaction, much more long term work will have to carried out before they will be applied in practice. Interesting work has been done on the activation of slags using admixtures but a more exhaustive work has to be carried out in North America for large scale applications. Corrosion inhibitors are being used to counteract the corrosion of the reinforcement. The long term corrosion aspects and also the possibility of using dual inhibitors are expected to receive more attention in the coming years. Improved properties are obtained by the use of polymers in concrete and further improvements and the possibility of new applications will be sought by using novel types of polymers in combination with other admixtures. Extended set retarders have been used for various purposes, such as in hot weather concreting, in concretes containing large amounts of cement, for long hauls, in large pours and for preventing cold joints. Work is expected to be carried out on the development of more efficient retarders that will not have any adverse effects on concrete.

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TABLE 1: PROPERTIES OF CONCRETE CONTAINING MODIFIED LIGNOSULFONATE AND OTHER SUPERPLASICIZERS*

Cem	Admixture	W/C	Slump mm	Compr Str. MPa days 7	28	91
I	None	0.49	75	26.8	32.8	37.8
I	Melamine-based	0.40	80	37.3	44.0	48.5
I	Naphth-based	0.40	80	36.3	42.6	49.9
V	None	0.48	90	19.1	32.2	38.0
V	Melamine-based	0.38	80	33.0	42.0	48.5
V	Mod-Lignosulf	0.38	85	32.8	42.4	50.3

* From Reference 1

TABLE 2 : FREEZING AND THAWING DURABILITY OF CONCRETE CONTAINING A LIGNOSULFONATE-BASED SUPERPLASTICIZER*

Sample	Slump (mm)	Rel Dynm Modulus %	Weight Loss %
Control	100.0	94	3.2
Lignosol-Modified	100.0	98	3.2
Control	195.0	87	2.2
Lignosol-Modified	208.0	53	4.5

* From Reference 2

TABLE 3 : INITIAL SET AND COMPRESSIVE STRENGTH OF CONCRETE CONTAINING SUPERPLASTICIZERS*

Admixture	Dosage (30% w/w)	W/C	Initial Set (h)	24h Comp Str MPa
Reference	-	0.65	5.50	10.3
Melamine-based	2.67	0.65	7.75	11.2
Naphth-based	2.00	0.65	9.25	11.35
Na-Lignosulf	1.50	0.65	12.50	6.0
Na Lignosulf + Accelerator	1.50	0.65	11.50	9.8

*From Reference 3

TABLE 4: INFLUENCE OF SYNTHESIZED LIGNOSULFONATE ON MORTAR*

Additive	% solid	Air %	Flow	Set Time h	min	Water-redn. (70%flow)
None	-	10.8	48	3	30	--
Melamine-Formaldehyde	0.3	11.0	80	4	30	11.3
Lignosulfonate	0.3	10.8	87	6	00	12.8

* (Bialski, A.M., and Nayak, K.V., Private Communication)

TABLE 5: RELATIVE WATER REDUCING CAPABILITIES OF SUPERPLASTICIZERS AT DIFFERENT DOSAGES*

Dosage (%) =	1	2	3	4	5
			Water Redn (%)		
Commercial	9	19	29	36	38
Synthesized I	13	25	35	38	41
Synthesized II	13	24	34	37	40

* From Reference 5

TABLE 6: STRENGTH GAIN IN CONCRETE CONTAINING ANTIFREEZING ADMIXTURES*

Admixture	Amount (%)	Percent of designed strength		
		7 days	28 days	90days
SN	4-6	30	70	90
CNNC	3-5	40	80	100
CC+SC	2-3	35	80	100

* From Reference 26

TABLE 7: STRENGTHS OF CONCRETE CONTAINING SOME ANTIFREEZING ADMIXTURES (EXPRESSED AS % OF 28 DAY STRENGTH AT 20°C)*

Admixture Type	Ordinary Cement	Rapid Hardening
Sod nitrate + Sod sulfate	85%	70%
Sod nitrite + Cal chloride	63%	63%
Pot carbonate + Retarder	74%	48%
Cal nitrate + Sod. sulfate	75%	43%

* From Reference 28

TABLE 8: CONCRETE WITH FREEZE-PROTECTION ADMIXTURE*

	PLAIN	NEW ADMIXTURE
Time of set (-4°C)	11 hr 55 min	5 h 45 min
	Strength (MPa)	
3d, -4°C	3.4	9.24
7d, 10°C	8.3	39.3
28d,10°C	18.1	49.9

* From Reference 29A

TABLE 9: THE EFFECT OF PARTICULATES ON SPACING FACTOR AND COMPRESSIVE STRENGTH*

LOW WATER/CEMENT

Sample	W/C	Concen., %	Spacing Factor mm	Comp.Str. MPa (28d)
Brick	0.60	4.33	0.58	31.3
Vermiculite	0.63	0.55	0.32	28.9
Pumice I	0.63	2.15	0.43	29.4
Pumice II	0.63	4.21	0.43	25.1
Perlite	0.63	0.51	0.43	25.2
Plain Conc.	0.60	----	----	31.3

TABLE 9 (cont'd.)

HIGH WATER/CEMENT

Sample	W/C	Concen., %	Spacing Factor mm	Comp.Str. MPa(28d)
Brick	0.68	8.66	0.29	30.2
Vermiculite	0.76	1.10	0.16	20.5
Pumice I	0.69	4.30	0.2	25.1
Pumice II	0.66	8.42	0.22	21.7
Perlite	0.64	1.02	0.20	24.4
Plain Conc.	0.60	----	----	31.3

* From Reference 38

TABLE 10: TEST RESULTS OF CONCRETE CONTAINING POLYMER AND GLUTARALDEHYDE AUXILIARY AGENT*

Polymer (%)	Glutaraldehyde (%)	Trans-mittance**	Slump (mm)	Strength (28 d) MPa
0.6	--	92	130	34.5
	0.5	95	125	34.8
	1.0	97	125	35.0
0.8	--	95	180	33.5
	0.5	98	170	34.0
	1.0	100	160	34.2
1.5	--	99	225	33.2
	0.2	100	220	33.8
2.0	--	100	220	32.0
	0.1	100	215	32.5
3.0	--	100	145	29.0
	0.1	100	140	29.2

* From Reference 69

** Measured as a turbidity by photoelectric photometry

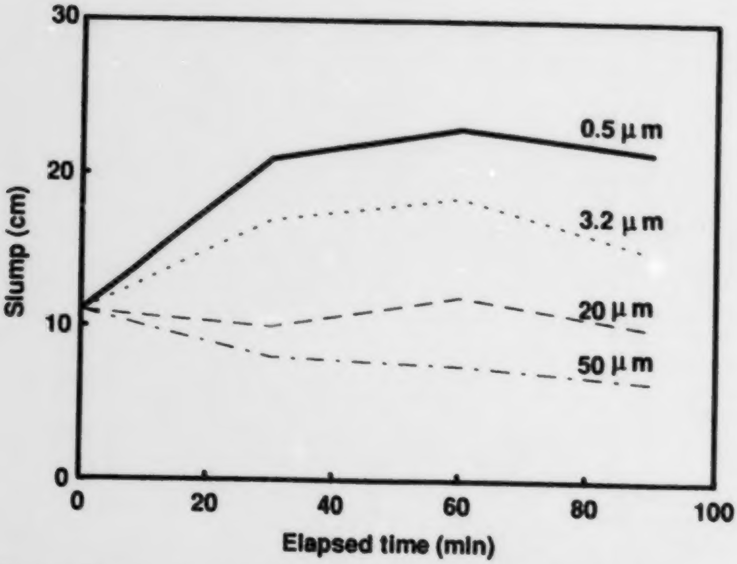

Fig. 1. Dependence of particle size of polymer on slump retention (From Ref. 15).

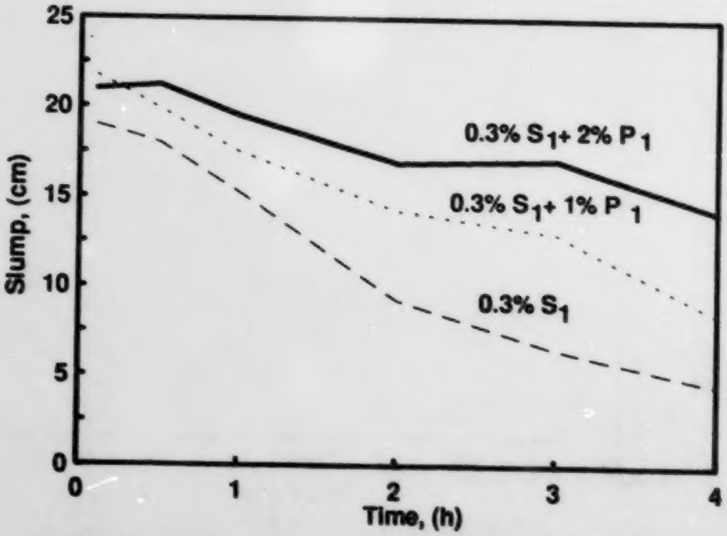

Fig. 2. Effect of commercial superplasticizer and polymer on slump (From Ref. 18).

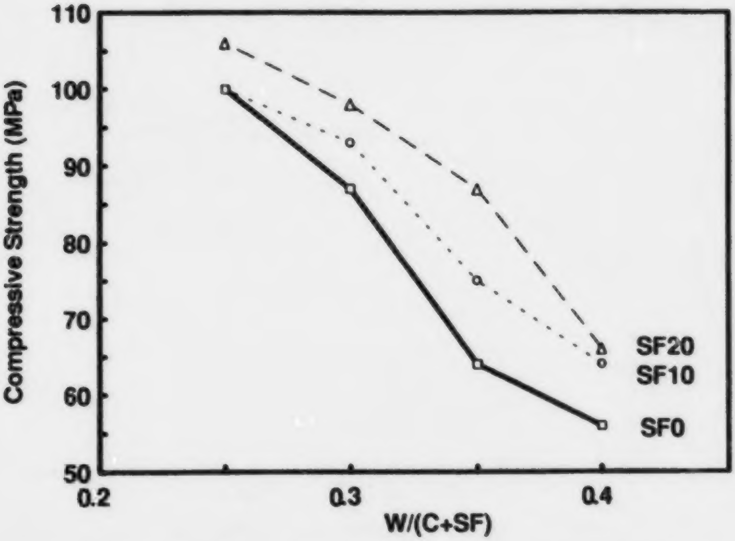

Fig. 3. Effect of silica fume on compressive strength development (From Ref. 21).

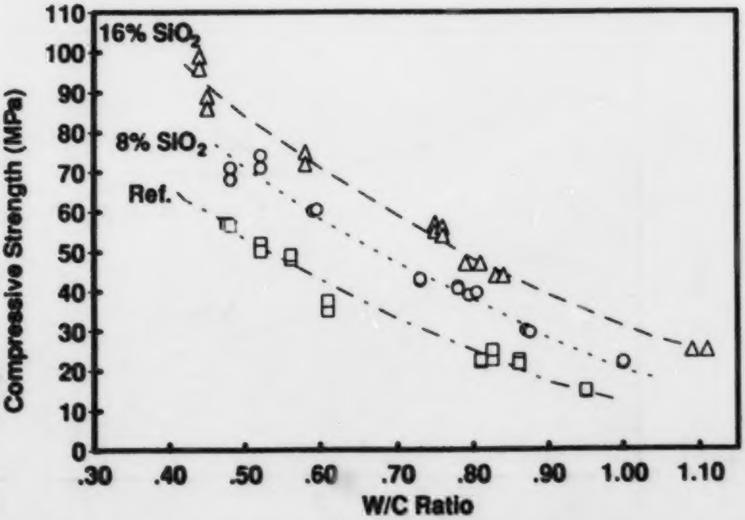

Fig. 4. Effect of silica fume on strength of concrete at different W/C ratios (From Ref. 22).

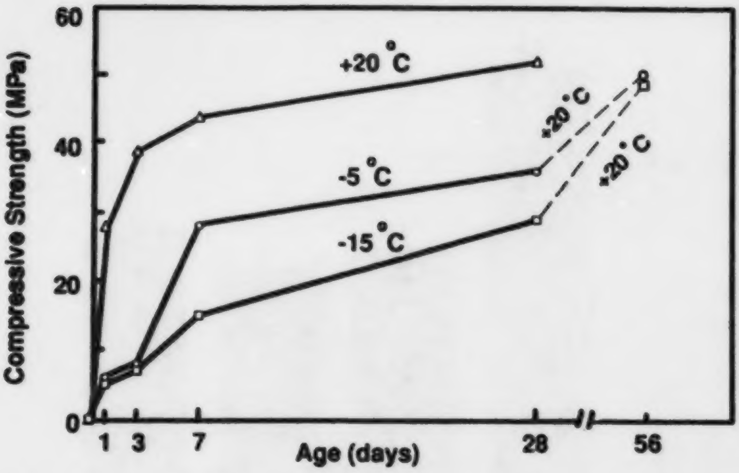

Fig. 5. Strength development of dry-mix mortar at low temperature. (From Ref. 27)

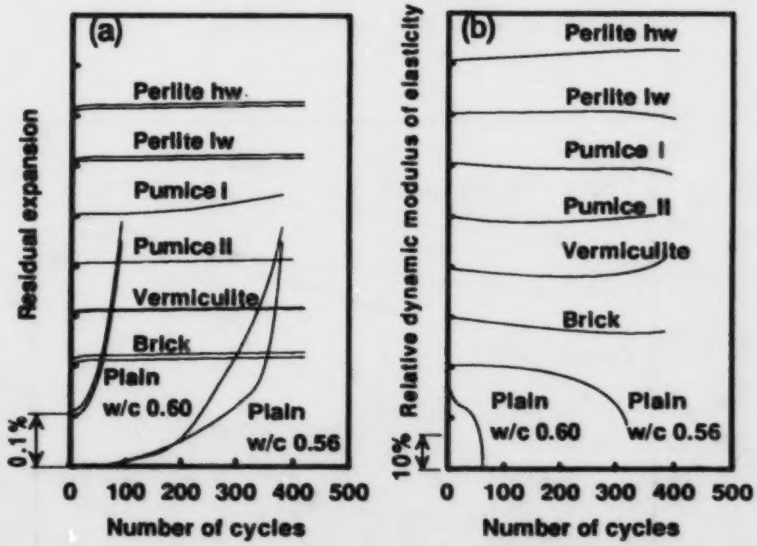

Fig. 6. Concrete with and without particulate admixture (a) residual expansion and (b) relative dynamic modulus of elasticity as a function of freeze-thaw cycles (hw = high W/C ratio; lw = low W/C ratio). (From Ref. 39)

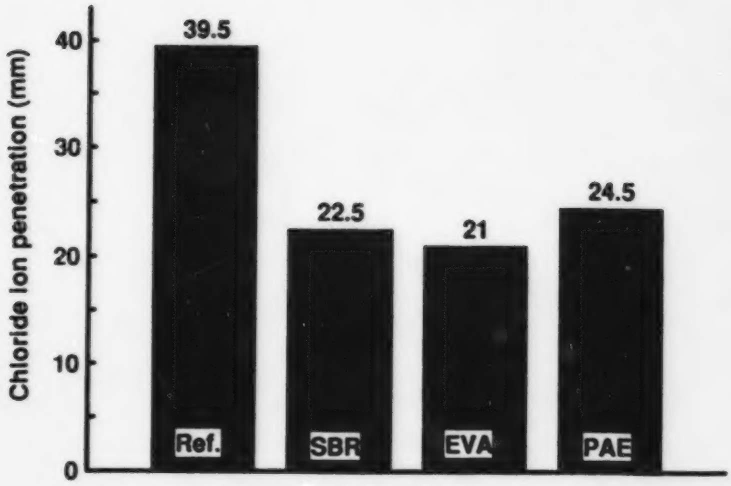

Fig. 7. Chloride ion penetration in polymer-modified concrete (From Ref. 43).

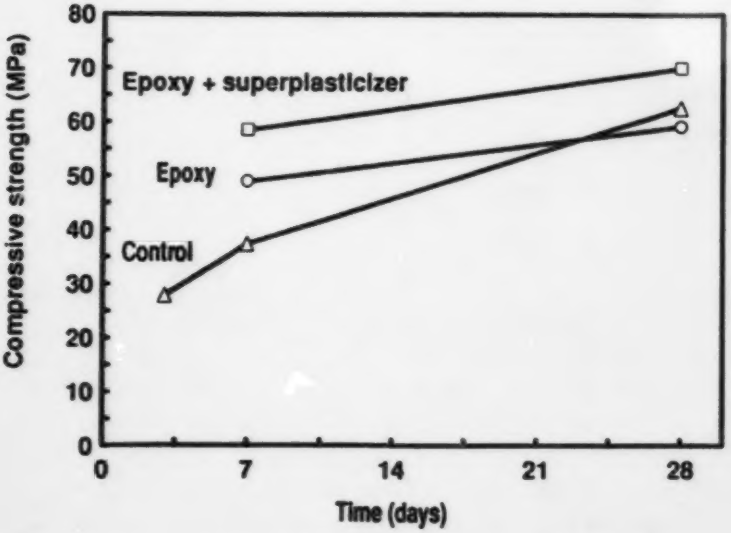

Fig. 8. Compressive strengths in epoxy-modified concretes (From Ref. 45).

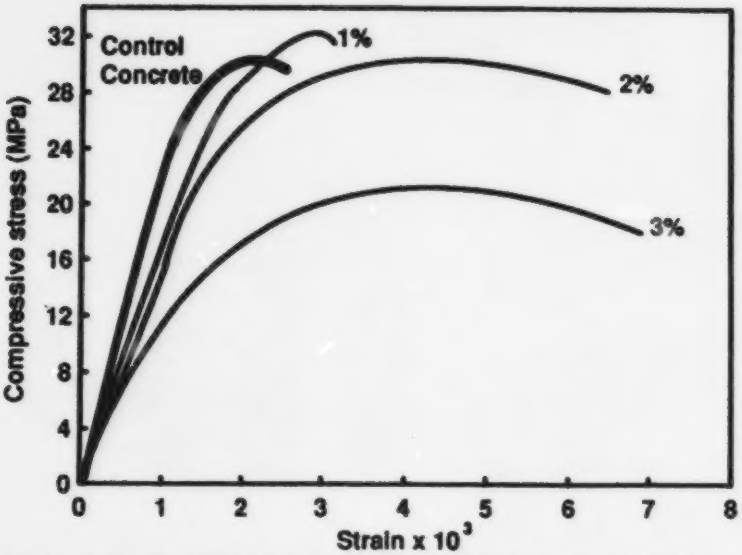

Fig. 9. Stress-strain behaviour of concrete containing superplasticizer and different amounts of rubber latex (From Ref. 47).

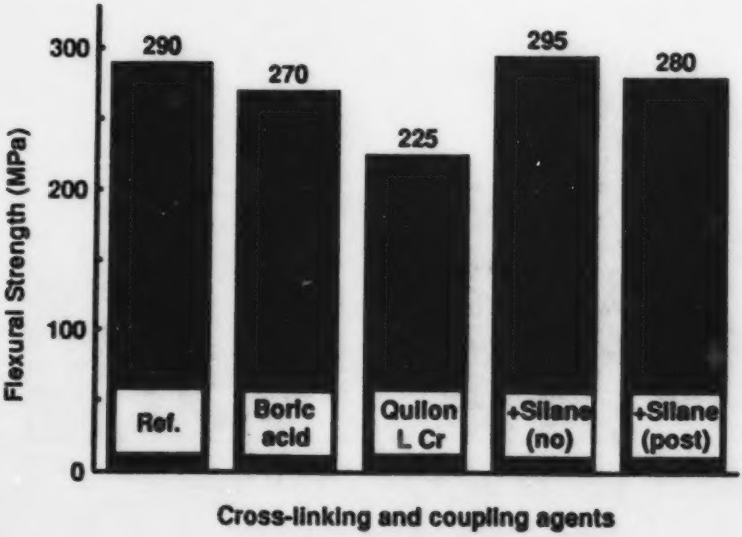

Fig. 10. Flexural strengths of MDF cements (Ref. 51).

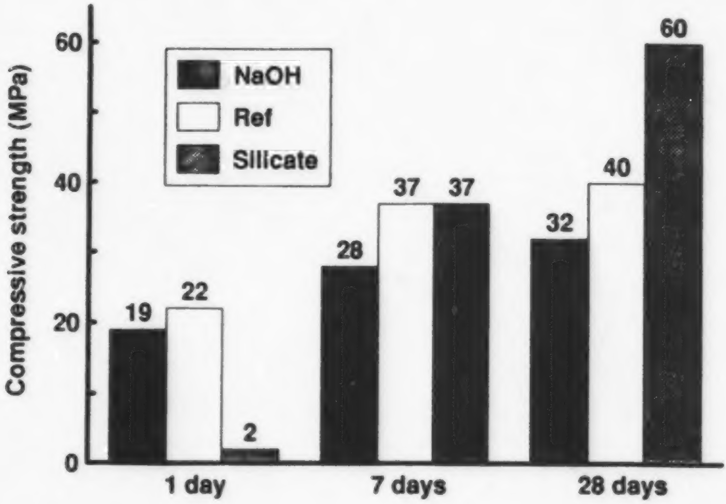

Fig. 11. Effect of activators on compressive strength of slags (From Ref. 60).

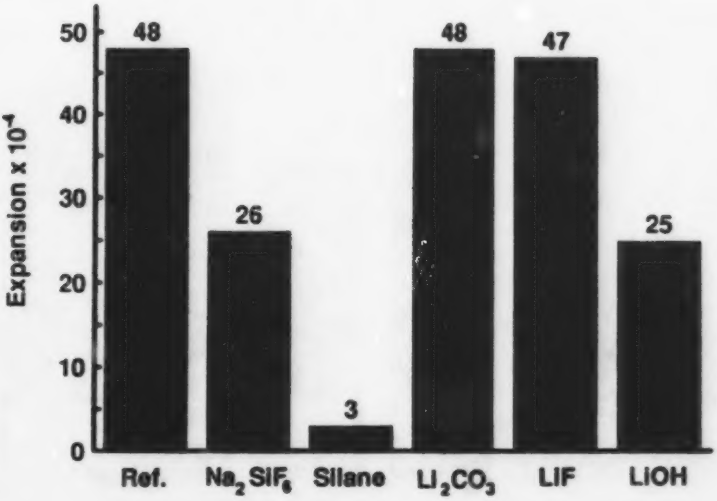

Fig. 12. Effect of chemicals (0.5%) on alkali-aggregate expansion (From Ref. 62).

WORLD-WIDE REVIEW OF CORROSION INHIBITORS IN CONCRETE

by Neal S. Berke and Thomas G. Weil

Synopsis: Corrosion of steel in reinforced concrete has caused the early failure of structures in environments where concrete is subjected to ingress of chlorides or to excessive carbonation. Several methods of corrosion protection are currently being used to extend the life of reinforced structures subjected to chloride environments. Commercially used methods include low permeability concrete produced with high-range water-reducing admixtures (HRWR), silica fume, epoxy-coated steel, calcium nitrite corrosion inhibitor, cathodic protection, latex modified concrete, and sealers. In numerous cases combinations of the above have been successfully used such as silica fume and calcium nitrite or calcium nitrite and epoxy-coated steel.

In this paper the use of inhibitors to provide corrosion protection is reviewed. This is supplemented with a world-wide review of commercial projects using inhibitors. The advantages are that the inhibitor is distributed throughout the concrete protecting all the steel, and that concrete's low permeability prevents the inhibitor from being lost. A review of early research on inhibitors for steel in concrete is given and it is shown how the search for an inhibitor that did not adversely affect concrete's properties led to the application and use of calcium nitrite.

Keywords: Concrete, Corrosion Inhibitors, Steel Reinforcement, Carbonation, Chloride Ingress, Review, Calcium Nitrite, Silica Fume, Concrete Resistivity, Chloride Permeability

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INTRODUCTION

Steel-reinforced concrete is one of the most widely used construction materials throughout the world. The alkaline environment provided by the concrete protects the steel from corrosion. However, when concrete is subjected to chlorides or excessive carbonation, or both, the normally passive steel can begin to corrode. The corrosion products have 2 to 4 times the volume of the steel and the stresses produced cause the concrete to crack. Eventually, delamination and spalling occur and the integrity of the concrete structures is lost.

One means of protecting reinforcing steel from corrosion is to add a corrosion inhibitor to the concrete. In this paper a review of the use of corrosion inhibitors in concrete is presented. Corrosion inhibitors used in concrete must not have adverse effects on the concrete properties in both the unhardened and hardened states. While numerous inhibitors have been suggested, only a small group have been seriously studied, and only one (calcium nitrite) is used commercially on a wide scale. This is partially based upon the fact that calcium nitrite improves the properties of hardened concrete and its accelerating effect is well within the range acceptable for ASTM C 494 admixtures. This accelerating effect is easily offset using commercially available ASTM C 494 retarders.

Therefore, after giving a brief mention of other suggested inhibitors, the paper will focus on those inhibitors studied in great detail, and in particular on calcium nitrite, which is in commercial use in the United States, Japan, and Europe. It will be shown that the use of calcium nitrite as a corrosion inhibitor is very effective in protecting steel in concrete. Its commercial use throughout the world is also reviewed.

REVIEW OF PRE-1980 STUDIES

The use of inhibitors in concrete has been reviewed by Griffin (1), Graig and Wood (2), Treadaway and Russell (3), Slater (4), and by Berke (5, 6). With the exception of Berke's reviews the previous reviews were written on work performed before 1980. Early studies looked at numerous inhibitors with the most attention focused on sodium nitrite, potassium chromate, sodium benzoate, and stannous chloride. This paper presents a brief review of early work and an update and expansion of the latest papers, including several new references.

Craig and Wood (2) examined the mechanical properties of mortars produced with sodium nitrite, potassium chromate, sodium benzoate, and calcium chloride. They found a marked decrease (as high as 20 to 40%) in compressive strengths when these inhibitors were added to the mortars. In contrast, calcium chloride increased the compressive strength. Tensile strengths were adversely affected by sodium nitrite and sodium benzoate, but not by potassium chromate.

Corrosion testing showing that these inhibitors, with the exception of sodium nitrite, did not work in the presence of chloride. Sodium benzoate and 2% potassium chromate were found to be detrimental in the presence of chloride. Results are summarized in Table 1.

Treadaway (7) and Treadaway and Russell (3) conducted tests of sodium nitrite and sodium benzoate. They also found a decrease in compressive strength when these chemicals were added to concrete. Corrosion tests were conducted in alkaline solutions and in cement extracts. In the cement extract experiments they found that sodium nitrite inhibited corrosion in the presence of chlorides, whereas sodium benzoate did not. In their solution testing they emphasize that corrosion was uniform with sodium benzoate and pitting occurred with sodium nitrite. Initiation of corrosion was delayed with sodium nitrite, with the delay increasing with inhibitor content.

Solution testing involved the aeration of unbuffered samples over several weeks (i.e. the solution did not maintain its initial protective concrete-like properties) and the solutions were most likely carbonated. In such a case uniform corrosion would be expected for benzoate and pitting would still be the mechanism of attack for nitrite. In good quality concrete, where carbonation is unlikely at the reinforcement level, the mechanism of corrosion in the presence of chloride will be pitting (7, 8). Pitting is a highly localized form of corrosion. Thus, the sodium nitrite inhibitor did not change the corrosion process into one of pitting, and did protect the steel until carbonation took place.

Early reviews (1, 2) mention the work of Arber and Vivian (10, 11) with stannous chloride, and indicate that it is a corrosion inhibitor. In examining the work of Arber and Vivian, one finds that stannous chloride appeared to be a noncorrosive set accelerator. Stannous chloride, in short term testing with and without admixed chlorides, and no further chloride exposure from the environment, did appear to decrease corrosion as measured by tensile strength regression and steel appearance (10) (mild corrosion and tensile loss still occurred). Stannic chloride on the other hand did not behave as an inhibitor. Arber and Vivian (10) attribute the improved corrosion resistance to the stannous ion, however, some of it is most likely due to a decrease in chloride content, since stannous chloride is 37% chloride, stannic chloride is 54% chloride, and calcium chloride is 64% chloride. Recent tests by Hope and Ip showed that stannous chloride was not a corrosion inhibitor (12).

Based upon the inhibitor experiments up to the mid 1970's it is not surprising that inhibitors were not in large scale use in concrete. Sodium nitrite was identified as a good inhibitor, but it caused severe strength loss. Sodium or potassium salts of chromates and benzoates gave mixed results on inhibition and also reduced strength. Stannous chloride increased strengths, but still contained chloride and was relatively expensive. Concrete technicians were in need of an inhibitor that was not detrimental to concrete's properties, improved corrosion resistance conclusively, and was affordable.

CALCIUM NITRITE CORROSION INHIBITOR

The commercial production of calcium nitrite as a stable solution of up to 40% dissolved solids encouraged the use of calcium nitrite as a corrosion inhibitor. Unlike sodium nitrite in which the sodium cations are detrimental to strength and could promote alkali-aggregate reaction, calcium nitrite was found to be nondetrimental to mechanical properties. For example, calcium nitrite meets ASTM C 494 specifications as a concrete accelerator (13). Calcium nitrite is not detrimental to concrete's properties, unlike the inhibitors mentioned above based on sodium or potassium. Later studies showed that calcium nitrite is compatible with silica fume (14, 15).

Laboratory Corrosion Testing

In the 1970's numerous corrosion studies were performed documenting the corrosion-inhibiting properties of calcium nitrite (13, 16, 17, 18). It was determined that the mechanism of corrosion protection was that of anodic inhibition, and that steel in chloride containing concrete fails by pitting. This was later documented by basic research in saturated calcium hydroxide solutions (9). Fig. 1 from reference 9 shows the effect of calcium nitrite in increasing the pitting potential (E_p) and nucleation potential (E_n, or the breakdown potential, E_b), which relate to increasing the difficulty of pitting initiation and growth.

The first large scale results reported by the United States Federal Highway Administration (FHWA) were released in 1983 (19). Even though the FHWA used a water-to-cement (w/c) ratio that was high (>0.5), and added chlorides to the mixing water, they concluded that "...calcium nitrite can provide more than an order of magnitude reduction in the corrosion rate." The final recommendation of this work was that calcium nitrite was effective at chloride to nitrite ratios below 1.0 based on admixed chlorides, and ratios "...higher than 1 might be tolerable..." for chloride ingress into initially chloride-free concrete (20). A recent review on calcium nitrite showed that for nonadmixed chlorides protection ratios are above 1.0 for chloride levels up to 9.5 kg/m^3 (16 lb/yd^3) (21). Solution experiments by Hope and Ip indicate that the ratios of chloride to nitrite can be higher than 11 to one (12).

The South Dakota Department of Transportation (DOT) conducted a study in which reinforcing bars were embedded in concrete cylinders with and without admixed chlorides (22). Cylinders with admixed chlorides went into corrosion immediately (1 day) unless calcium nitrite was present. Samples with calcium nitrite remained passive for 700 days. Calcium nitrite also improved the corrosion resistance of samples without admixed chlorides. Chloride analyses at the reinforcing bar showed that calcium nitrite prevented corrosion at chloride-to-nitrite ratios of 1.6 to 2.2 (chloride contents exceeded 10.7 kg/m^3 or 18 lb/yd^3), as shown in Table 2 (21).

Fig. 2 shows an updated curve for corrosion versus time first presented in 1987 (23). After four years of accelerated corrosion only specimens with calcium nitrite remained uncorroded, at chloride levels at the steel exceeding 7 kg/m^3 (12 lb/yd^3) (24). It is clear that under long-term accelerated testing calcium nitrite is an effective corrosion inhibitor, and that the better quality of the concrete the better the corrosion inhibition. This is further documented in another large scale study completed in 1987, in which the corrosion rates of minislabs were investigated (14).

Additional data supporting the effectiveness of calcium nitrite corrosion inhibitor has come from Japan (25, 26). In accelerated tests with wetting and drying cycles at 80°C, calcium nitrite was found to be an effective inhibitor (25). Table 3 summarizes the results showing cycles to corrosion as a function of chloride and inhibitor contents (33% solids by mass of solution).

In spite of the high water-to-cement ratios (w/c) calcium nitrite still proved effective as seen in Table 3. Another test of wet/dry cycling at 50°C demonstrated the advantages of using a lower w/c of 0.5 versus 0.6 as shown in Fig. 3 (25).

Nishibayashi et al (26) document long-term performance of a "rust inhibitor," identified as calcium nitrite in their presentation, as a corrosion inhibitor in field marine exposures. They showed that at the 20 L/m^3 (4 gal/yd^3) addition level of a 30% solution, calcium nitrite reduces corrosion.

Recent experiments by Tomosawa et al (27) also document the effectiveness of a 33% calcium nitrite solution in accelerated laboratory testing of steel-reinforced concretes. In addition to tests on admixed chloride at 80°C under wet and dry cyclic exposure, they studied chloride penetrating from cyclic immersion in 3% NaCl at 50°C. Calcium nitrite was found to be very effective in improving corrosion resistance as shown in Table 4 taken from their paper.

Though most of the work with calcium nitrite has looked at the protection against chloride ions, work by Andrade et al(28) shows that it also protects steel in carbonated concrete.

Increasing use of pozzolans to prevent corrosion by reducing permeability to chlorides is taking place. Silica fume is one of the most effective pozzolans because its small particle size enables it to act as a filler and increases its chemical reactivity. Studies have shown that it improves the corrosion performance of fly ash concretes (21) and concretes with slag (26). Figure 4 from Berke and Rosenberg (21) shows improvement in the corrosion resistance of embedded steel in concrete containing fly ash when calcium nitrite is used.

Calcium nitrite is compatible with silica fume (14, 29) and should provide protection in the presence of chloride. Fig. 5 from Berke et al (19) shows the benefits of combining both silica fume and calcium nitrite. Fig. 6 which is based upon data in this study shows the significant increase in corrosion that occurred for a silica fume containing concrete without calcium nitrite. This occurred even though the AASHTO T277 rapid chloride permeability was under 400 coulombs for the non-nitrite specimens (29). The specimens with calcium nitrite had decreased resistivity and increased permeability numbers, however, they remained passive (29). Previous studies showed that even though permeability numbers increased and resistivities decreased, chloride ingress was not increased (14, 15).

In addition to protecting steel in concrete recent testing shows that calcium nitrite protects also galvanized steel and aluminum (30). This study involved the alternate ponding of minibeams with 3% NaCl solutions, and the measurements of corrosion by the macrocell and polarization resistance methods. Fig. 7 shows the reduction in total corrosion for aluminum conduit in concrete exposed to chlorides, and is based on data in Berke et al (30).

Calcium nitrite has been shown to provide benefits in cracked concrete (21, 31, 32). Fig. 8 shows the long-term total corrosion (measured by polarization resistance and integrated) for reinforced notched minibeams with cracks, produced by third point loading, that extend down to the reinforcing bar, half-way to the bar, and not at all. Cracked samples with calcium nitrite took longer to go into corrosion or were corroding at substantially reduced rates. Cracks were less than 0.01 inch in width (21). Visual analysis showed that both the depth of attack as well as the extent of surface attacked were significantly reduced by the addition of calcium nitrite (31). More recently a study of unnotched beams cracked in flexure showed that calcium nitrite provided enhanced performance to chloride induced corrosion (32).

A German study (33) showed that sodium nitrite was also effective in reducing corrosion in cracked specimens, even at much higher than recommended water-to-cement ratios and low dosage rates of inhibitor. Other investigators have shown in general that shallow, narrow cracks were not a significant factor in corrosion (34, 35).

Extensive research shows that calcium nitrite will perform well in cracked concretes. This improved performance is due to several factors. One is that, in concrete produced to proper ACI guidelines and codes, crack sizes at the surface will be less than 0.2 mm (0.008 inch) and cover will be at least 1.5 inches. Under these conditions the crack size at the steel will be quite small and the solution will be buffered by the concrete.

Studies have shown that calcium nitrite might decrease the resistivity of concrete (15, 23). However, long-term corrosion data show that in spite of the decrease in resistivity, corrosion rates were significantly reduced (15, 23, 24, 29). Likewise, increases in AASHTO T277 chloride permeability values with calcium nitrite were clearly shown to have no negative effects on actual chloride ingress into the concretes (14, 15). Indeed, as pointed out above, silica-fume concrete with calcium nitrite outperformed that without, even though resistivities were decreased by calcium nitrite (29).

Laboratory Studies on Nitrite Stability

Several tests were performed to determine the nitrite content of concrete with calcium nitrite as a function of time and environment. Analysis of the nitrite contents in the concretes studied by the South Dakota DOT (22) are given in Table 4. The nitrite was determined by the FHWA method PB-275428 and chloride analysis by the Florida DOT Research Report 203 PB-289620 method. These results show that nitrite at the reinforcement level remained high after 4 years of severe chloride exposure. Furthermore, they show that the nitrite protected the steel against high chloride levels.

Another test was performed on concrete cylinders (100 mm in diameter) containing 2% calcium nitrite by mass of cement exposed to a fog room five years or to 2% NaCl for five years. The w/c was 0.44 and the cement factor was 375 kg/m^3 (634 lb/yd^3). Nitrite and chloride analyses were performed as described above and are given in Table 2. While some nitrite is lost at the surface there is no loss in the interior where the reinforcement would be placed. This is not unexpected since the surface typically has a higher porosity than the bulk concrete, and because the surface was in continuous contact with the solution. The moist-cured specimens had no loss of nitrite at any level indicating that nitrite is extremely stable in concrete.

Tomosawa et al (27) also showed good stability of nitrite in concrete. The above coupled with the good corrosion performance in numerous long-term tests, indicates that loss of nitrite is not a concern in good quality concrete with proper concrete cover over the steel reinforcement.

MECHANISM FOR CALCIUM NITRITE CORROSION PROTECTION

The Mechanism by which calcium nitrite protects steel in concrete from chloride induced corrosion was addressed recently by Berke and Rosenberg (21) and Berke and El-Jazairi (31).

The corrosion inhibition by calcium nitrite could be outlined on the basis of the Pourbaix (36) potential-pH equilibrium diagram, and other studies (9, 37-43).

The first reaction that occurs when steel is placed in an alkaline environment, such as concrete, is:

$$Fe \rightarrow Fe^{2+} + 2e^{-} \qquad (1)$$

Later reactions will convert the Fe^{2+} to $Fe(OH)_2$, or $Fe_3O_4 \bullet nH_2O$, or Y-FeOOH as passive oxide layer at the steel surface. In the absence of chloride all the oxide phases are stable in alkaline environments. These oxide layers are only a few monolayers thick since once they are formed they prevent further oxidation from occurring. Y-FeOOH is the most stable in the presence of chloride or other depassivating ions.

On the microscopic scale there will be regions where the protective oxide is not present. At these locations chloride ions can form a complex with Fe^{2+}. The resulting complex can migrate from the steel surface and subsequently convert into an expansive corrosion product.

Calcium nitrite prevents the continuation of the corrosion reaction by reacting with the ferrous ions, Fe^{2+}(18).

$$2Fe^{2+} + 2OH^{-} + 2NO_2^{-} \rightarrow 2NO\uparrow + Fe_2O_3 + H_2O \qquad (2a)$$

or

$$Fe^{2+} + OH^{-} + NO_2^{-} \rightarrow NO\uparrow + \text{Y-FeOOH} \qquad (2b)$$

If corrosion reaction (1) occurs, the ferrous ions produced are changed through calcium nitrite to the stable passive layer. Thus, the passivation chemical reaction of nitrite with ferrous ions is to block active corrosion centers by producing a passive ferric oxide protective film. The iron in the ferric state is not complexed by chloride and corrosion is reduced.

The nitrite does not enter into reactions involved in producing the anode, but reacts with the resulting products of the anode. Thus it cannot affect the size of the anode. Also only monolayers of oxides are involved, essentially no nitrites or hydroxide are consumed in forming the initial protective oxides or hydroxides, which is supported by the nitrite concentration data at the reinforcement level given in the previous section.

Both Cl^- (or other aggressive anions), NO_2^-, and OH^- are competing at "flaws" in the oxide for Fe^{2+}. Over time, nitrite or an alkaline environment free of chloride or both will reduce the number of sites where Fe^{2+} ions are formed through formation of a protective coating Y-FeOOH or $Fe_2O_3 \cdot nH_2O$.

Recent studies as part of the Strategic Highway Research Program add new evidence supporting the outlined mechanism of protection. These studies showed that nitrite modifies the oxide film on the reinforcing bar to be more protective than that which naturally occurs in concrete (44). They also demonstrated that this improved oxide leads to enhanced corrosion resistance (45).

When chloride is present at extremely high $[Cl^-]/[NO_2^-]$ or $[Cl^-]/[OH^-]$ ratios or both the probability of Cl^- complexing a Fe^2 cation is increased. This leads to $[Cl^-]/[NO_2^-]$ or $[OH^-]$ protection ratio beyond which pitting would be likely. The ratios for $[Cl^-]/[OH^-]$ are typically stated to be between 0.27 to 0.57 (46). In Berke and Rosenberg (21) an analysis of hundreds of concrete and solution tests was performed showing that the $[Cl^-]/[NO^-2]$ is approximately 1.5 to 1, as shown in Fig. 9.

PROPER DESIGN

It is important that the civil engineer do more than just add an inhibitor to the concrete. ACI 201 states that low water-to-cement ratios and good concrete cover should be employed when chloride exposures are moderate to severe. Indeed, several of the studies mentioned emphasized the need for low water-to-cement ratios and adequate cover (14, 23, 25). Furthermore, it is well documented that calcium nitrite becomes more effective as concrete quality improves (14, 15, 21, 23, 24). Since higher quality concretes outperform those of lesser quality, improvements with calcium nitrite are substantial.

In addition to following ACI guidelines for cover and w/c it is also recommended that ACI 224R be followed for maximum crack widths. The published studies showing improvements in cracked concretes with calcium nitrite were at crack widths essentially at or below maxima recommended in ACI 224R.

If the concrete is to be subjected to freezing and thawing cycles then it must properly air entrained according to ACI 201. Berke and Roberts (15) showed that concretes with calcium nitrite and HRWA can be properly air entrained and resistant to freezing and thawing.

FIELD USE OF CALCIUM NITRITE

The first commercial use of calcium nitrite as a corrosion inhibitor in concrete was in Japan to offset the high chloride contents in marine sand. In this use there is over 10 years of successful experience. (27)

The commercial use of calcium nitrite as a corrosion inhibitor for ingressed chlorides started in 1978 with bridge deck applications on an experimental basis by various state departments of transportation in the Unites States. These older bridge decks in Illinois, New Hampshire, and Maine contain calcium nitrite protection for uncoated steel reinforcement. Several have recently been cored and show that the nitrite is still present in original quantities and corrosion measurements indicate complete passivity. Table 6 gives calcium nitrite levels and chloride levels in specimens taken from a deck in Illinois. These results are in good agreement with the laboratory data.

In 1980 a small commercial fish pier on Cape Cod, Massachusetts was installed using calcium nitrite as its only corrosion protection system in the prestressed concrete. A 1991 examination of this fish pier revealed that there is no sign of corrosion. The excellent condition of these older structures confirm the benefits of calcium nitrite as has also been demonstrated in the long-term laboratory studies.

Based upon the extensive laboratory studies and initial field successes the use of calcium nitrite is growing. Over two hundred parking structures, in the United States and Canada, from Halifax to Boston to Seattle, now contain calcium nitrite. Some protect only uncoated steel reinforcement and prestressed strands, others are in conjunction with epoxy-coated reinforcement, and in others the concrete contains silica fume, fly ash, and granulated blast-furnace slag. The use with silica fume is a growing trend.

Two of the oldest parking structures are St. Anthony's and MTC Gateway in Minneapolis which were completed in 1981 and 1983 respectively. St. Anthony's contains 6,100 m^3 (8,000 yd^3) while MTC Gateway contains 9,200 m^3 (12,000 yd^3) of concrete. The original concrete mixture used 17.5 L/m^3 (3.5 gal/yd^3) of a 30% calcium nitrite solution. Cores taken in the spring of 1991 from both structures found chloride levels between 2.7 to 7.2 kg/m^3 (4.6 to 12.1 lb/yd^3) of concrete at the reinforcement level, 38 mm (1.5 inches deep). Nitrite amounts were determined and found to be essentially unchanged at the reinforcement level. The specified w/c was 0.45 and there is no sign of corrosion or deterioration at this time.

The earliest marine structure was completed in 1980. Since then over fifty structures have been completed. These include bridges, seawalls, piers, docks, and repair work. Owners range from state departments of transportation, to city and country municipalities, private industry, and the Unites States Navy. The largest marine structure was the 11,500 m^3 (15,000 yd^3) pier for the Port of Long Beach, California completed in 1990. The state of North Carolina completed the mile-long, 8,400 m^3 (11,000 yd^3) Roanoke Sound Bridge in the spring of 1990. North Carolina requires calcium nitrite for all prestressed concretes in marine environments. After the 50-year old Jamestown Bridge failed due to reinforcement corrosion, the state of Rhode Island is including calcium nitrite in the prestressed concrete trestle portion for the replacement bridge. A large shipbuilder in Groton, Connecticut uses calcium nitrite to repair old deteriorated pier piles.

The biggest note of confidence of calcium nitrite's performance record comes from a number of state departments of transportation in the United States who use calcium nitrite in prestressed and precast concrete bridges girders in a deicing salt environment. Steel girders require sand blasting and repainting every ten to twenty years. This is costly and time consuming and often interrupts traffic. States are looking for low-maintenance, long-life solutions to the bridge deterioration problem. Illinois (since 1980), Michigan and Ohio (since 1984), and Maine (since 1987) combine the excellent qualities of prestressed and precast concrete AASHTO and box girders with the corrosion protection capabilities of calcium nitrite to obtain the low-maintenance, longer design-life structures that they require. Over 230,000 m^3 (300,000 yd^3) of concrete have been placed in concrete girders throughout the Unites States.

The first major European use of calcium nitrite in a bridge structure occurred in Northern Italy in 1988. This prestressed bridge is part of the New Trafori Motorway and used 7,500 m^3 (10,000 yd^3) of concrete containing 15 L/m^3 of 30% calcium nitrite solution. The cement factor was 420 kg/m^3 (710 lb/yd^3) and the w/c was 0.45. The concrete contained a high-range water reducer to produce a 200-mm (8-inch) slump. The 3-day compressive strengths were 31 to 32 MPa (4500 to 4700 psi) and the 28-day compressive strengths were 61 to 63 MPa (8900 to 9100 psi).

Calcium nitrite has been used in other concrete structures where chloride induced corrosion is a threat. These include salt manufacturing plants, salt-water cooling towers, brine sludge tanks, salt bath test facilities, truck delivery areas, loading docks, deicing salt storage domes, truck weigh stations, underground utility vaults and run-off water culverts (to protect aluminum-coated steel-culvert). All signs point to continued growth in the number of applications for this corrosion inhibitor for metal in concrete.

CONCLUSIONS

Only one inhibitor is in large scale commercial use and it is calcium nitrite. A history of its commercial use showed that it is used in numerous applications throughout the world. Calcium nitrite is successfully being used because:

- It provides corrosion inhibition in the presence of chlorides.
- It is not detrimental to concrete properties.
- It is available in sufficient quantities for commercial use in concrete.

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Table 1
Visual Observations of Craig and Wood (Reference 2)

$CaCl_2$ (percent)	CHEMICAL COMPOUND	PERCENT	PERCENT OF SURFACE CORRODED 0	20	40	60	80	100
0			o-					
2				o-				
4					o-			
0	$NaNO_2$	2	o-					
0	$NaNO_2$	4	o-					
2	$NaNO_2$	2	o-					
2	$NaNO_2$	4	o-					
0	$NaC_7H_5O_2$	4	o-					
0	$NaC_7H_5O_2$	6	o-					
2	$NaC_7H_5O_2$	4			o			-
2	$NaC_7H_5O_2$	6			-			o
0	K_2CrO_4	2	o-					
0	K_2CrO_4	4	o-					
2	K_2CrO_4	2			o-			
4	K_2CrO_4	4		-	o			

Note: o-specimen cured in air; -=specimen cured in water

$NaNO_2$ = Sodium Nitrite
$NaC_7H_5O_2$ = Sodium benzoate
K_2CrO_4 = Potassium chromate

Table 2
South Dakota DOT Test Samples After 4 Years of Cyclic Ponding in 9% NaCl (From References 21, 22)

Initial Nitrite Content 5.44 kg/m³
Cement Factor = 367 kg/m³, w/c=0.43

POSITION (STEEL DEPTH)	NITRITE CONTENT (kg/m³)	CHLORIDE CONTENT (kg/m³)	$[Cl^-]/[NO_2^-]$
Bottom	5.38	11.83	2.20
Middle	6.75	10.53	1.56
Top	6.80	11.30	1.66

Note that steel was not corroded. Polarization resistance value was 230 kΩ-cm².

Table 3
Number of Wet Dry Cycles at Which Cracking Occurs
(Reference 25)

NaCl Level	Corrosion Inhibitor L/m^3 (gal/yd^3)			
% Concrete	0	11.8 (2.4)	23.5 (4.7)	35.3 (7.1)
0	0	-	-	-
0.1	0	0	-	-
0.3	10	15	0	-
0.5	10	10	15	0

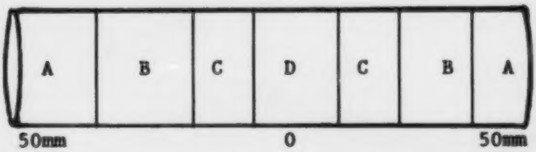
w/c = 0.6; 0 = Did not crack. Corrosion inhibitor was 30% calcium nitrite solution.

Table 4
Effect of 30% calcium nitrite solution and w/c on chloride level that induced corrosion (from reference 27).

30% Calcium Nitrite (L/m^3)	0		10		20		30	
W/C	0.5	0.6	0.5	0.6	0.5	0.6	0.5	0.6
NaCl content (%)	0.06	0.11	0.24	0.29	0.32	0.43	0.49	0.45
Days to Corrosion Initiation	88	60	242	134	278	192	355	234

Table 5
5 Year Nitrite and Chloride Analysis for Cylinders in 2% NaCl or Fog Room -375 kg/m³ Cement and 0.44 w/c
(A-outside, D-center)

Core through the midpoint of a 100-mm diameter cylinder.



SAMPLE	LOCATION	PONDED 2% NaCl Cl (kg/m³)	CN	% CN s/s	% CN s/s Fog room
1	A	2.5	3.3	0.9	2.0
	B	2.6	6.7	1.8	
	C	2.7	7.3	1.9	
	D	2.4	8.2	2.2	2.6
2	A	5.2	5.1	1.4	1.7
	B	3.6	6.1	1.6	
	C	3.7	7.5	2.0	
	D	2.7	6.9	1.9	2.3

Table 6
Illinois Bridge Deck Nitrite and Chloride Levels
After 7.5 Years

Original Nitrite Content (as $Ca(NO_2)_2$) was approximately 2200 ppm

SAMPLE	DEPTH (mm)	NITRITE (ppm)
1A	13-25	2320
1B	44-57	2726
2A	13-25	1924
2B	44-57	2735

There is no corrosion after 7.5 years.

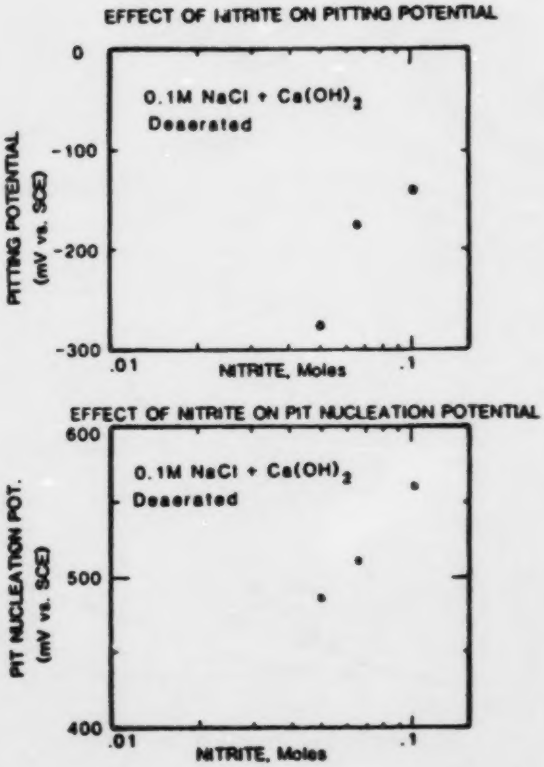


Fig. 1 - Effect of nitrite concentration on E_n and E_p of 1045 steel in deaerated saturated $Ca(OHO_2{}^n$+ 0.1 M NaCl (Fig. 7 from reference 9).

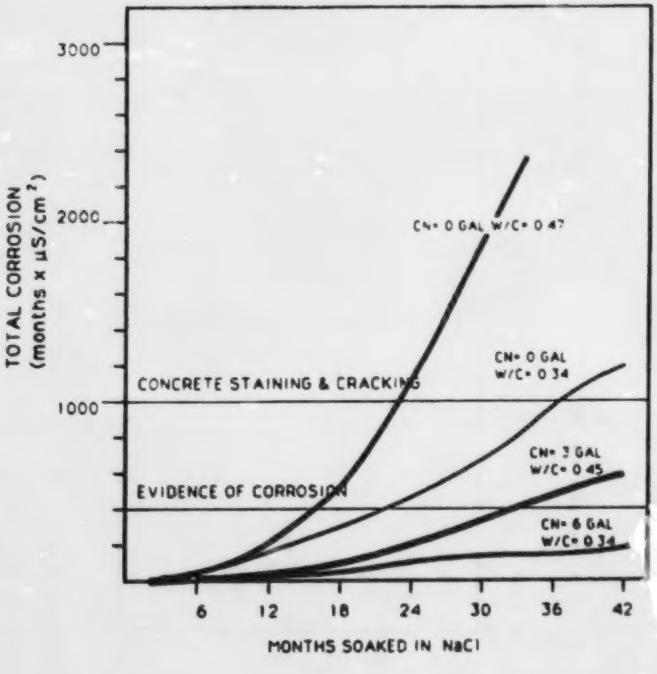


Fig. 2 - Total corrosion versus time as a function of 30% calcium nitrite addition and w/c ratio (Fig. 2 from reference 14).

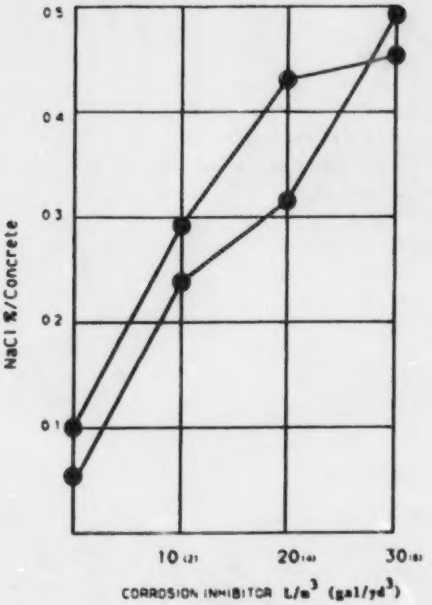


Fig. 3 - Effect of w/c ratio, inhibitor content, and salt content on corrosion of steel in concrete (from reference 25).

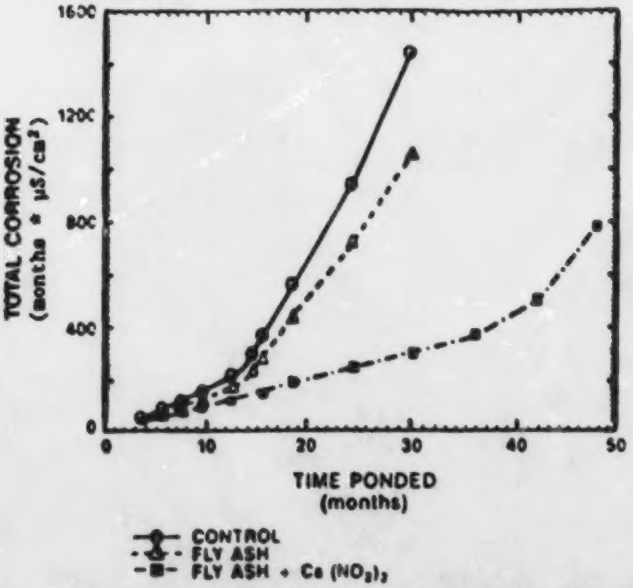


Fig. 4 - Effect of fly ash and fly ash plus calcium nitrite on the total corrosion of reinforcing bars in concrete ponded with 3% NaCl (Fig. 4 from reference 21).

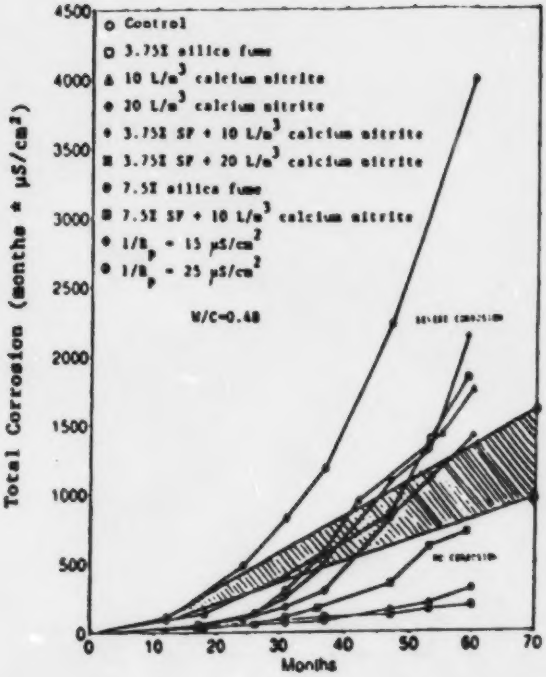


Fig. 5 - Total lollipop corrosion as a function of calcium nitrite and silica fume content (Fig. 1 from reference 29). Note that calcium nitrite improves the corrosion resistance of concretes containing silica fume.

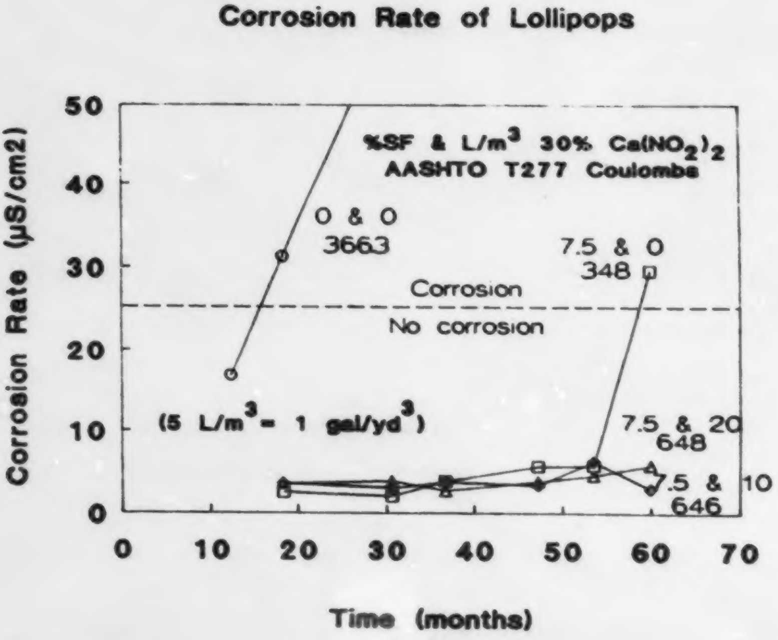


Fig. 6 - Corrosion rates of lollipops as a function of calcium nitrite content with 7.5% silica fume by mass of cement based on data used in reference 29. Note that even though calcium nitrite increases AASHTO T277 Rapid Chloride Permeability, it significantly improved corrosion resistance. 1/Rp values greater than 25 µS/cm^2 are indicative of severe corrosion.

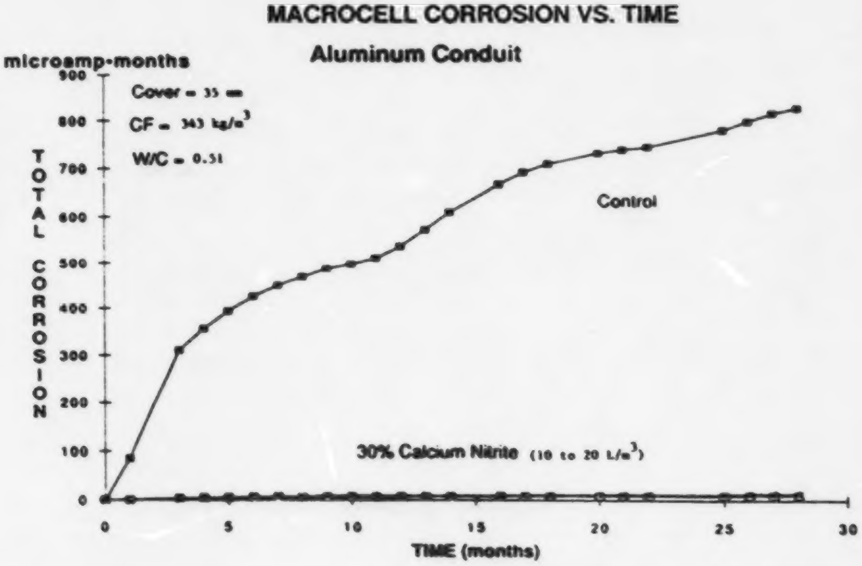


Fig. 7 - Macrocell corrosion versus time from aluminum conduit in concrete exposed to cyclic 3% NaCl ponding as a function of calcium nitrite content. Curve is from the integrated data in reference 30.

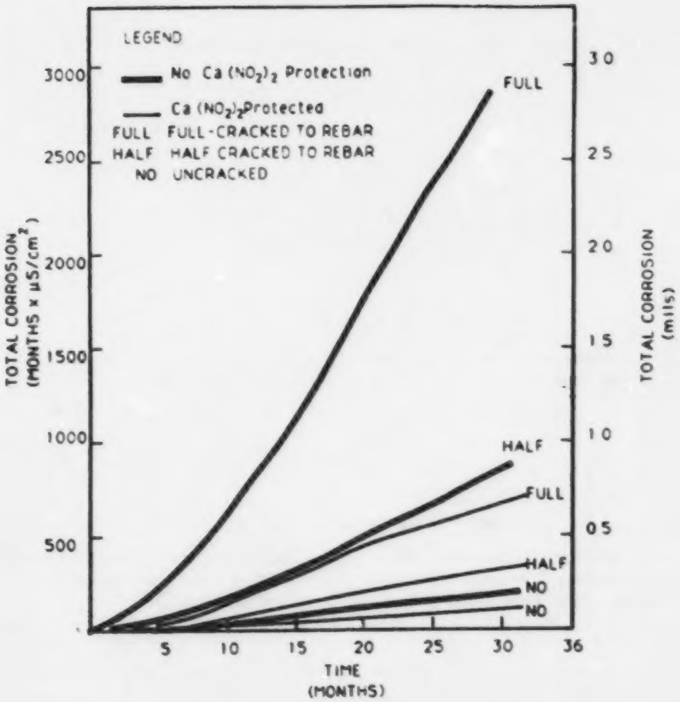


Fig. 8 - Total corrosion versus time for cracked concrete with and without calcium nitrite (Fig. 3 from reference 21).

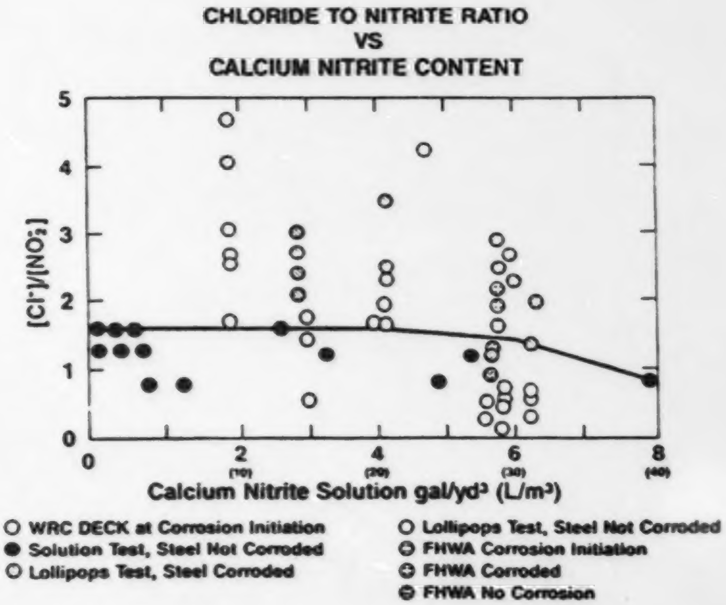


Fig. 9 - Chloride-to-nitrite ratio at which corrosion begins versus calcium nitrite content (Fig. 5 from reference 21).

ROLE OF SILICA FUME IN CONCRETE: A REVIEW

by

V.M. Malhotra, G.G. Carette and V. Sivasundaram

ABSTRACT

This review paper outlines the production and properties of silica fume, and describes its role in concrete. The properties of fresh and hardened concrete incorporating silica fume are described. The durability aspects of silica fume concrete are discussed including its performance under exposure to freezing and thawing conditions, chemical attack and de-icing salt conditions. Reference is made to the use of silica fume as a mineral admixture to control alkali-aggregate reactions. The paper is concluded by outlining the applications of silica fume concrete, specifications for the material, and a list of pertinent references.

Keywords: abrasion, alkali aggregate reactions, chemical attack, chloride ions, concrete,compressive strength, creep, de-icing salt scaling, drying shrinkage, flexural strength, freezing and thawing, heat of hydration, modulus of elasticity, setting time, silica fume, specifications.

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INTRODUCTION

Mineral admixtures such as fly ash and granulated blast-furnace slag are being used increasingly in concrete. These admixtures not only reduce the amount of cement to be used but also enhance many durability characteristics of concrete such as resistance to the ingress of chloride ions and sulphate attack. In recent years silica fume, a by-product in the production of metallic silicon or ferrosilicon, has gained acceptance for use as a mineral admixture in concrete.

The original investigations for the use of silica fume in concrete were performed in Norway in the early 1960's. The research organizations in Canada and the U.S.A. initiated investigations in the 1980's. The first international conference on the use of silica fume in concrete was held in Montebello, Canada in 1983, followed by the conferences in Madrid, Spain in 1986, and in Trondheim, Norway in 1989, and the next conference is being held in Istanbul, Turkey in 1992.

This paper presents the state-of-the art on the properties of silica fume and its use in concrete. Also discussed are the durability aspects of silica fume concrete, and the current status of the standards on the subject.

PRODUCTION OF SILICA FUME

Silica fume is a by-product of the manufacture of silicon or of various silicon alloys produced in submerged electric-arc furnaces. The type of alloy produced and the composition of quartz and coal, the two major components of the burden of the submerged-electric arc furnace, influence greatly the chemical composition of silica fume (1). Most of the published data on the use of silica fume in cement and concrete are related to silica fume collected during the production of a silicon alloy containing at least 75% silicon.

FORMS OF SILICA FUME

There are several forms in which silica fume is available commercially both in North America and in Europe. The available forms are as follows.

As-Produced Silica Fume

This refers to silica fume as collected in a dedusting system known as bag houses. In this form the material is very fine with a bulk loose density of about 200 to 300 kg/m^3 compared with 1500 kg/m^3 for portland cement (1). As-produced silica fume is available in bags or in bulk. Because of its extreme fineness, this form does pose handling problems; in spite of this, the material can be and has been transported and handled like portland cement.

Compacted Silica Fume

This form of silica fume has a bulk loose density ranging from 500 to 700 kg/m^3 and is considerably easier to handle than the "as produced" silica fume. In order to produce the compacted form, the as-produced silica fume is placed in a silo, and compressed air is blown in from the bottom of the silo. This causes the particles to tumble and in doing so they agglomerate. The heavier agglomerates fall to the bottom of the silo from where they are removed at intervals. This air compaction of the as-produced silica fume is so designed, that the agglomerates produced are rather weak, and quickly break down during concrete mixing. Mechanical means have also been used to produce compacted silica fume.

Water-Based Silica Fume Slurry

In order to overcome the handling and transporting problems associated with 'as produced silica fume', several companies are marketing water-based slurries of silica fume, with slurries containing about 40 to 60% of solid particles. Typically, these slurries have a density of about 1300 kg/m^3. Some slurries may contain chemical admixtures such as superplasticizers, water reducers and retarders. One such product known as Force 10,000 has been successfully marketed in North America.

Pelletized Silica Fume

Disc pelletizers have been used to pelletize as-produced silica fume, and the process involves the use of small amounts of water (1). The pellets so produced are rather strong and do not break down easily during concrete mixing. Because of this, the use of pelletized silica fume has not gained much acceptance in the concrete industry. One cement manufacturer in Canada is producing pelletized silica fume, and this product is then interground with portland cement clinker to produce silica fume blended cement*.

PHYSICAL AND CHEMICAL CHARACTERISTICS OF SILICA FUME

Physical Characteristics of Silica Fume

Silica fume varies in colour from pale to dark grey. The carbon content, and to a lesser extent, the iron content seems to have preponderant influence on the colour of silica fume. The loose bulk specific weight is of the order of 200 kg/m^3 and 500 kg/m^3 when compacted.

Its specific gravity is about 2.20. The particles have a wide range of sizes but they are perfectly spherical (Fig. 1). The mean particle diameter is 0.1 μm compared with 10 μm for particles of cement. The specific surface of silica fume ranges from 13,000 to 30,000 m^2/kg as measured by the nitrogen adsorption technique, the value for portland cement being 300 to 400 m^2/kg as measured by the Blaine method. A typical particle size distribution for silica fume is shown in Fig. 2 (1).

Table 1 gives values of fineness, specific surface, pozzolanic activity index and specific gravity for silica fume from the production of silicon and ferrosilicon alloy (1).

X-ray diffractograms of samples of different types of silica fume show them to be vitreous (1). All the diffractograms exhibit a very wide scattering peak centred at about 4.4Å, the most important peak of cristobalite (Fig. 3).

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* At present, several cement companies in Canada are marketing silica fume blended cements, and these are rapidly gaining acceptance.

When heated to 1100°C, silica fume crystallizes in the form of cristobalite, except FeSi-50% type, which crystallizes as enstatite, most probably due to the presence of a high amount of iron and magnesium oxide (Fig. 4).

Chemical Composition of Silica Fume

The chemical composition of silica fume from different furnaces is given in Table 2 (1). A change in the type of alloy manufactured may cause changes in the characteristics of the corresponding silica fume produced. Therefore, it is important that concrete plants using silica fume should know of any changes in the source of the raw materials used for the furnace or changes in the nature of the alloy being produced by a plant.

PHYSICAL AND CHEMICAL MECHANISMS IN THE CEMENT-SILICA FUME SYSTEM

Physical Mechanisms

There are several physical mechanisms by which silica fume contributes to the enhancement of properties of concrete. These include the increased strength of the bond between the paste and aggregate by reducing the size of the CH crystals in the region (a): by providing nucleation sites for the CH crystals so that they are smaller and more randomly oriented (b): by reducing the bleeding and increasing the cohesiveness of the mixtures, and (c): by reducing the thickness of the weaker transition zone (2,3). They also include the increased density of composite system by the filler packing effect, and by providing a more refined pore structure (4,5,6). For the above mechanisms to take place, it is essential that silica fume particles be well dispersed in a concrete mixture. To achieve this, the use of high-range water reducers (superplasticizers) becomes almost mandatory.

Chemical Mechanisms/Pozzolanic Reactions

Malhotra et al. (1) have reported extensive information on the chemical reactions involved in the cement-silica fume-water system. In the presence of portland cement, the basic reaction, known as the pozzolanic reaction, involves the combining of finely divided amorphous silica with lime to form a calcium silicate hydrate.

Grutzeck and co-workers (7,8) have developed a mechanism for the pozzolanic reaction of silica fume with $Ca(OH)_2$ in aqueous solutions. It is based on the analysis of ions in solution, and application of other methods such as infra-red, X-ray, and SEM. According to this mechanism, when in contact with water, silica goes into solution and within 1 hr or less, the silica in solution forms an amorphous silica-rich, Ca-poor gel on the surface of silica fume particles. The silica fume particles also form agglomerates. With time, this silica-rich, Ca-poor coating dissolves and the agglomerates of silica fume start reacting with $Ca(OH)_2$ to form C-S-H. The initial reaction described above is surface dependent. The formation of C-S-H by the pozzolanic reaction is temperature dependent.

In the presence of cement, the above simple mechanism becomes more complex and is described by Grutzeck et al. (8) as follows:

> "In this model silica fume comes into contact with water containing low concentrations of lime and absorbs moisture to form a silica-rich gel consuming most of the available water. The gel particles clump together in the interstices of the cement particles and agglomerate into large masses. In contrast to the network of radiating C-S-H crystals formed in normal cement pastes, in C/S pastes the anhydrous calcium silicate grains are surrounded by the development of silica gel within 15 min to 1 hr. At 3 hr, CH in the pore solution of normal cement pastes forms discrete crystals, but in the presence of silica fume no such crystals of CH form. The excess lime reacts with the outer surface of the agglomerated gels to produce additional C-S-H which intergrows with the hydrating silicate grains. At 24 hr and later, C-S-H development is enhanced in the silica fume pastes, and a matrix of C-S-H cemented agglomerates of silica fume that are intergrown with C-S-H growing outward or inward from the calcium silicate grains is formed. This results in the formation of a more rigid structure at earlier ages than that occurring in normal cement pastes."

This model is based on density, compressive strength and chemical analysis, and is used to explain early strength, lower permeability, density and thickening that are observed in cement pastes containing silica fume.

The high pozzolanicity of silica fume has been reported by Buck and Burkes (9) and others (10-12). In their research, Buck and Burkes found that during the hydration of 1:1 mixtures of silica fume and CH, all CH was consumed by 28 days.

PROPERTIES OF FRESH CONCRETE

Colour

In general, the colour of silica fume concrete is darker than that of the conventional concrete. This colour difference is more evident on the surface of wet hardened concrete and in fresh concrete. Also, in concretes incorporating high levels of silica fume having high carbon content, this dark colour is more pronounced. In investigations of silica fume concrete made with dark coloured fume, it has been observed that this colour difference disappeared when concrete specimens were stored in the laboratory environment for extended periods (13). Presumably, this difference in colour has been neutralized by drying and perhaps by carbonation.

Water Demand

Owing to its spherical, small sized particles, silica fume fills the pores between larger grains of cement and gives a better particle size distribution, leading to a decrease in water demand in silica fume concrete. However, on the other hand, the high specific area of the silica fume particles tends to increase the water demand, giving a net effect of increased water demand compared to a portland cement concrete with the same level of workability. Figure 5 shows that the increase in water demand is almost directly proportional to the amount of silica fume used in concretes having an initial water-to-cement ratio of 0.64 (14). Superplasticizers or high-range water reducers can be used to advantage in silica fume concretes in reducing the water demand. In superplasticized silica fume concretes, the water demand decreases at all dosages of silica fume because of the dispersing action of the superplasticizer.

Markestad (15) observed that the reduction in water demand resulting from the incorporation of water reducing admixtures in silica fume concretes was directly proportional to the admixture concentration in the mixing water, rather than to the amount of silica fume. He proposed a mixture-proportioning method based on an efficiency factor of a given admixture.

Bleeding

Bleeding of silica fume concretes is generally lower than that of plain portland cement concrete. The extremely fine silica fume particles attach themselves to the cement particles, reducing the channels for bleeding and leaving very little free water available in the fresh concrete for bleeding (8,16,17,18). Figure 6 shows the significant reduction in bleeding with increasing silica fume replacement (19).

Workability

When compared to the portland cement concrete, silica fume concrete is more cohesive and resistant to segregation due to the increase in the number of solid-to-solid contact points. Silica fume concrete also tends to lose slump rapidly, and a higher initial slump than that of a conventional concrete is often required (20). However, in lean concretes incorporating less than 300 kg/m^3 cement, the workability appears to improve by the addition of silica fume.

Ivanov and Roshavelov (21) reported that when 7.5 to 15% by weight of cement is replaced by silica fume, the yield stress and apparent viscosity increased in cement/silica fume pastes. However, at low replacements, high reductions in yield stress and apparent viscosity were measured. The small amount of silica fume in the paste improved the packing of cement and silica fume grains resulting in reduced yield stress and apparent viscosity.

Air Entrainment

Because of the extremely high surface area of the silica fume, the dosage of air-entraining admixture required to produce a certain volume of air in silica fume concretes increases considerably with increasing silica fume dosage (Fig. 7). The presence of carbon in the silica fume adds to the increase in air-entraining admixture demand. It has been reported that entrainment of more than 5% air is difficult in concretes incorporating high amounts of silica fume, even in the presence of a superplasticizer (14). Also, it was reported that the air-entraining admixture dosage increased significantly with silica fume content at only low water-to-cementitious material ratio when comparing concretes made with various water-to-cementitious material ratios (Fig. 8) (22).

Shrinkage Cracking

Shrinkage cracking occurs in fresh concrete under curing conditions which cause a net removal of water from exposed concrete surfaces, thus creating tensile stresses beyond the low early age tensile strength capacity of concretes. As concretes containing silica fume show little or no bleeding, thus allowing very little water rising to the surface, the risk of cracking is high in fresh concretes. Shrinkage cracking can be a very serious problem under curing conditions of elevated temperatures, low humidity, and high winds, allowing rapid evaporation of water from freshly placed concrete. Johansen (23) and Sellevold (24) reported that fresh concrete is most vulnerable to shrinkage cracking as it approaches initial set. To overcome this problem, the surface of concrete should be protected from evaporation by covering of plastic sheets, wet burlap, curing compounds, and evaporation retarding admixtures.

Setting Time

Investigations show that ordinary concrete mixtures (with 250 to 300 kg/m^3 cement) incorporating small amounts of silica fume, up to 10% by weight of cement, exhibit no significant difference in setting times compared to conventional concretes (25). As silica fume is invariably used in concretes in combination with water reducers and superplasticizers, the effect of silica fume on setting time of concrete is masked by those of the admixtures. The investigations by Bilodeau (19) have shown that the addition of 5 to 10% silica fume to superplasticized and non-superplasticized concretes had a negligible effect on the setting time of concrete (Fig. 9). However, in a concrete with water-to-cementitious material ratio of 0.40 and 15% silica fume, there was a noticeable delay in setting time. The high dosage of superplasticizer used due to the high silica fume content in the concrete could have contributed to the setting delay (Fig. 10) (19).

Heat of Hydration

Researchers have shown that in concretes incorporating silica fume as cement replacement, a reduction in adiabatic heat is observed with no decrease in strength (26). A study by Tachibana et al. (27) corroborating the previous work, showed that in high strength concrete incorporating 540 kg/m^3 cement and 10% silica fume, the adiabatic heat rise was 9% lower than that of a concrete without silica fume. It has been shown however, that the addition of silica fume accelerates the temperature rise during the first 72 hours compared to a similar portland cement concrete (28). The overall temperature rise in later ages was found to be lower than that of the reference concrete. Such steep initial temperature rise in the silica fume concretes has been attributed to the accelerating effect silica fume has on the cement hydration.

PROPERTIES OF HARDENED CONCRETE

Compressive Strength

It is well recognized that silica fume can contribute significantly to the compressive strength development of concrete. This is because of the filler effect and excellent pozzolanic properties of the material which translate into a stronger transition zone at the paste-aggregate interface.

The extent to which silica fume contributes to the development of compressive strength depends on various factors such as percentage of silica fume, water-to-cementitious materials ratio (W/C+SF), cementitious materials content, cement composition, type and dosage of superplasticizer, temperature, curing conditions and age.

Superplasticizing admixtures play an important role in ensuring an optimum strength development of silica fume concrete. The water demand of silica fume concrete is directly proportional to the amount of silica fume (used as a percentage replacement for portland cement) if the slump of concrete is to be maintained constant by increasing the water content rather than by using a superplasticizer. In such instances, the increase in the strength of silica fume concrete over that of control concrete is largely offset by the higher water demand, especially for high silica fume content at early ages, as illustrated in Fig. 11. In general, the use of superplasticizer is a prerequisite in order to achieve both proper dispersion of the silica fume in concrete, and fully utilize the strength potential of the fume. In fact, many important applications of silica fume in concrete depend strictly upon its utilization in conjunction with superplasticizing admixtures.

In early investigations, attempts were often made to establish the relative value of silica fume with respect to its strength producing properties by means of efficiency indexes. For example, the efficiency of silica fume was said to be 3, if 3 kg of cement could be replaced by 1 kg of silica fume with the resulting strength being the same as that of control concrete. Sellevold and Radjy (29), and Jahren (20) have reported values generally ranging between 2 and 5 for this index. Other investigators have reported even wider ranges, and this is not surprising in view of the so many factors that can affect the strength development of silica fume concrete. The efficiency factor as defined above is of limited usefulness because it generally does not take into consideration the different superplasticizer requirement of each type of concrete.

Silica fume concretes have compressive strength development patterns which are generally different from those of portland cement concretes. The strength development characteristics of these concretes are somewhat similar to those of fly ash concrete, except that the results of the pozzolanic reactions of the former are evident at earlier ages. This is due to the fact that silica fume is a very fine material with a very high amorphous silica content. The main contribution of silica fume to concrete strength development at normal temperatures takes place between the ages of about 3 and 28 days. The overall strength development patterns can vary according to concrete proportions and composition, and are also affected by the curing conditions.

Carette and Malhotra (30) have reported investigations dealing with the short- and long-term strength development of silica fume concrete under both conditions of continuous water-curing and dry-curing after an initial moist-curing period of 7 days. Their investigations covered superplasticized concretes incorporating 0 and 10% silica fume as a replacement by weight for portland cement, and W/C+SF ranging between 0.25 and 0.40. The compressive strength development at ages up to 3.5 years under continuously wet curing conditions is illustrated in Fig. 12, for concrete with a W/(C+SF) of 0.40. As expected, the major contributions of silica fume to the strength take place prior to 28 days with the largest gains in strength of the silica fume concrete over the control concrete being recorded at the ages of 28 and 91 days. However, this gain progressively diminished with age. For concretes with a W/C+SF of 0.30 and 0.40 it largely disappeared at later ages. Under air-drying conditions, the strength development pattern was found to be not significantly different from that of water-cured concretes up to the age of about 91 days; thereafter, however, air-drying clearly had some adverse effect on the strength development of both types of concrete. The effect was generally more severe for silica fume concrete where some reduction in strength was recorded between the ages of 91 and 3.5 years, especially for concrete with a W/C+SF of 0.30 and 0.40. These trends of strength reduction have not been clearly explained yet, but they appear to stabilize at later ages, and therefore are probably of little practical significance.

Curing temperatures have also been shown to affect significantly the strength development of silica fume concrete. This aspect has been examined in some detail by several investigators in Scandinavia. In general, these investigations have indicated that the pozzolanic reaction of silica fume is very sensitive to temperature, and elevated temperature curing has a greater strength accelerating effect on the silica fume concrete than on comparable portland cement concrete (4,31).

The dosage of silica fume is obviously an important parameter influencing the compressive strength of silica fume concrete. For general construction, the optimum dosage generally varies between 7 and 10%; however, in specialized situations up to 15% silica fume has been incorporated successfully in concrete.

Flexural and Splitting-Tensile Strength

The published data concerning the effect of silica fume on the flexural strength development of concrete indicate considerable variations in the results between different investigators (30,32-37). This is largely due to the different test procedures, and the size of test specimens used by different laboratories. Furthermore, as for portland cement concrete, the determination of flexural strength of silica fume concrete is particularly sensitive to variations in the moisture conditions of the specimens at the time of testing: and also due to the fact that the test has an inherently high variability.

In general, the available data indicate that the flexural strength development pattern of silica fume concrete under normal curing conditions is essentially the same as that for portland cement concrete (30,32-34). Based on Norwegian experience, Gjorv (35) concluded that the ratio of flexural to compressive strength was similar for both types of concretes at least within the range of structural applications. It was, however, pointed out that early drying out affects more the flexural strength of silica fume concrete than that of the portland cement concrete. CANMET's investigations have led to similar findings (30,36,37).

The limited published data on the splitting-tensile strength of silica fume concrete indicate that the ratio of splitting-tensile to compressive strength is also of the same order for both silica fume and portland cement concretes (34,38).

Bond Strength

The bond strength characteristics of silica fume concrete have been studied by a number of researchers, and there is a clear agreement that silica fume significantly improves the bond strength at the paste-aggregate as well as at the paste-steel reinforcement interfaces. In their study on the bond effects in high-strength silica fume concretes, Goldman and Bentur (39) concluded that silica fume has an inherent effect on strength enhancement, which increases with an increase in silica fume content. This effect was attributed to the improved aggregate-matrix bond resulting from the formation of a less porous transition zone in the silica fume concrete. The denser transition zone at the paste-aggregate interface in silica fume concrete has also been observed by several other investigators (40-45). Similarly, Gjorv et al. (46) reported that the presence of silica fume affects the morphology and microstructure of the steel-cement paste transition zone which was found to be reduced both in terms of porosity and thickness. The authors suggested that the improved steel-concrete bond in silica fume concrete is due to the reduced free water at the interface during casting, reduced preferentially-oriented CH crystals at the transition zone, and the densification of the transition zone. The enhanced bond characteristics at the paste-reinforcement interface has also been reported for lightweight silica fume concretes (47,48). Silica fume has also been reported to improve the bond between cement paste and various types of reinforcing fibres (4,49-51).

Modulus of Elasticity and Poisson's Ratio

Based on the published data by various investigators, there appears to be no significant differences between the Young's modulus of Elasticity "E" of concrete with and without silica fume (33,34,52,53). Malhotra et al. (52) have reported data on the Young's modulus of elasticity of portland cement/ blast-furnace slag/silica fume concrete (Table 3). They found that regardless of the various percentages of silica fume contents and water-to-cementitious materials ratios, there is no significant difference between the "E" values obtained at 28 days.

For portland cement concrete, it is known that the "E" values do not continue to increase with increasing compressive strength of concrete. At high strength levels, the stress-strain curves of concrete exhibit high fracture strains and steeper post-failure slopes, indicative of the more brittle nature of the material than at normal strength levels. This behaviour is equally true of high-strength silica fume concrete. Typical values of "E" for silica fume concrete in the compressive strength range from 80 to 100 MPa are of the order of 40 to 45 GPa (33,34,38). Poisson's ratio of silica fume concrete within the above strength range has been reported to vary between 0.20 and 0.25 (38).

Creep

The published data on the creep strain of silica fume concrete are sparse. Bilodeau et al. (53) reported an investigation on the mechanical properties, creep, and drying shrinkage of high-strength concretes incorporating fly ash, slag, and silica fume. The cementitious material content of the concretes was about 530 kg/m^3, and the water-to-cementitious material ratio was 0.22. In addition to the reference concrete, concretes with silica fume at 7% and 12% cement replacements, fly ash at 25% cement replacement, slag at 40% cement replacement, combination of 7% silica fume and 25% fly ash, and 7% silica fume and 40% slag were investigated. Specimens of these concretes were subjected to creep loading after 35 days of moist curing and at an applied stress equivalent to 35% of the compressive strength. The results obtained in this investigation are shown in Table 4. After one year duration, the creep strains of the reference, 7% silica fume, and 12% silica fume concretes were 1505×10^{-6}, 713×10^{-6}, and 836×10^{-6}, respectively. For concrete with 7% silica fume and 40% slag, the creep strain measured was the lowest at 641×10^{-6}. The pore structure and minimal quantity of free water in this concrete with the most amount of supplementary materials could have resulted in these very low creep values, at about 40% of the creep of the reference concrete.

From a study of the creep and shrinkage deformations of high-strength concretes with and without silica fume, de Larrard (55) reported that the autogenous shrinkage during setting is higher in silica fume concrete than in concrete without silica fume. Also, he noted that the creep strain of high-strength silica fume concrete is highly dependent on the age of concrete at loading.

In an investigation by Tachibana et al. (27), creep coefficients of a conventional concrete (W/C of 0.57) and high-strength silica fume concretes (W/C+SF of 0.22, 0.25, and 0.28) incorporating 10% silica fume were compared. After 180 days of creep loading, the creep coefficient of the normal strength concrete was approximately 2.60, whereas, that of the other three high-strength concretes was about 0.42. Limited data on creep strains of silica fume concrete have also been published by Wolsiefer (33) and Buil and Acker (54).

In summary, the studies performed up to now on high-strength silica fume concretes indicate that in general, these concretes exhibit lower creep strain as compared to the conventional concretes or high-strength portland cement concretes.

Drying Shrinkage

Investigations carried out on the drying shrinkage of silica fume concretes indicate that the long-term drying shrinkage of the concretes is not significantly affected by the silica fume addition. In an investigation carried by Johansen (32), silica fume concretes with silica fume content ranging from 0 to 25% and W/C+SF ranging from 0.37 to 1.06 were exposed to a drying environment (50% R.H.) at the ages of 1 and 28 days. It was found that when the W/C+F is lower than 0.60, the drying shrinkage was about the same for the reference concrete and the silica fume concrete. Some of these mixtures incorporated water reducers. However, the shrinkage values were found to be higher for concretes containing 25% silica fume and no water reducer. In another investigation, Pistilli et al. (25) monitored the drying shrinkage of silica fume concretes. At 64 weeks, the concrete with 237 kg/m^3 cement and a W/C+SF of 0.70 showed only a slight increase in drying shrinkage when 24 kg/m^3 of silica fume was incorporated, whereas for a concrete with 297 kg/m^3 cement and W/C+SF of 0.60, the drying shrinkage remained the same with or without silica fume.

The initial curing period and the conditioning of the test specimens before exposure to drying environment plays a critical part in the drying shrinkage of silica fume concrete. Wolsiefer (33) monitored the drying shrinkage strains of high-strength concretes made incorporating silica fume and high dosages of a superplasticizer. The concretes were moist cured for 1 and 14 days before the drying shrinkage measurements were made according to ASTM C 157. The drying shrinkage strains were found to be higher for specimens moist cured for one day. Maage (18) also reported that concretes containing silica fume exhibit higher drying shrinkage than the control concretes when these concretes were exposed to drying immediately after demoulding.

Figure 13 shows the drying shrinkage strain results of concretes made with a W/C+SF of 0.40 and incorporating 0, 15, and 30% silica fume, reported in an investigation by Carette and Malhotra (14). The concretes were subjected to 28-day initial moist curing before the drying shrinkage tests (ASTM C 157). After 420 days of drying, the control concrete and the concrete with 15%

silica fume underwent about the same drying shrinkage strains, while the concrete with 30% silica fume showed slightly lower values.

The drying shrinkage strains of normal strength concrete (W/C of 0.57) and high-strength silica fume concretes (W/C+SF of 0.22, 0.25, 0.28) containing 10% silica fume were monitored in a study by Tachibana et al. (27). At one year, the drying shrinkage strains of the high-strength concretes were in the range of 540 to $610x10^{-6}$, whereas, the shrinkage of the normal strength concrete was $930x10^{-6}$.

In an investigation reported by Bilodeau et al. (53), drying shrinkage tests were made on high-strength concretes after 7 days of moist curing. The concretes incorporated 0, 7, and 12% silica fume, as well as combinations of silica fume, fly ash, and slag. Figure 14 shows the composition and drying shrinkage strains of these concretes up to 448 days. Concretes containing silica fume, even in the presence of other supplementary cementing materials, showed lower drying shrinkage strains. The drying shrinkage strains of concrete incorporating 7% silica fume was found to be the lowest of all. De Larrard (55) explains the phenomenon of low drying shrinkage in high- strength concretes by pointing to the minimal free water in the system due to the low water-to-cementitious material ratio. Such concrete will lose its free water primarily through hydration, resulting in a higher autogenous shrinkage and low drying shrinkage. As silica fume is incorporated in high strength concrete, the pores in the hardened concrete are reduced in size, thus increasing the surface tensions, which results in increased autogenous shrinkage. Also, as the silica fume refines the pore size distribution of the hardened concrete, the permeability of the water vapour through the hardened paste is lowered, thus lowering the depth of drying at a given age (55).

DURABILITY ASPECTS

Permeability

Permeability of concrete is the one of the most critical parameters in the determination of the concrete durability to aggressive waters. As the permeability of concrete is lowered, its resistance to chemical attack increases. In a broader sense, permeability encompasses the transportation of liquid as well as gas and vapour phases through the concrete, and it is evaluated by measuring any of the following: weight loss of concrete over a period of drying, water transport under a pressure gradient, and oxygen and chloride ion diffusion in water saturated concrete.

The incorporation of supplementary cementing materials such as fly ash, slag, silica fume, and natural pozzolans in concrete results in fine pore structure and changes to the aggregate/paste interface, leading to a decrease in permeability. This decrease is much higher in concretes incorporating silica fume due to its high pozzolanicity. The filler and pozzolanic activity of the silica fume as well as the virtual elimination of bleeding improve the interfacial zone by pore refinement.

During investigations of the drying rates of 3-mm thick disks of cement pastes with and without silica fume, Sellevold (11) reported that the relative diffusion coefficient was reduced from 1.0 of the control paste to 0.25 for the paste with 20% silica fume. The author also reported another study showing the efficiency factor of silica fume in paste with respect to water diffusion during drying to be about 3, when comparing cement paste and pastes containing 8 and 16% silica fume as cement replacement. In a similar investigation made on concrete disks, Sorensen (56) found that the efficiency factor of silica fume to be ranging from 6 to 8, in concretes containing 8 to 16% silica fume in comparison with reference concrete. Thus, the incorporation of silica fume proves more beneficial in concretes, because of the improvements in the interfacial zone between the paste and the aggregate, than the cement paste phase itself.

A study by Manmohan and Mehta (57) showed that an addition of 10% by weight of highly reactive pozzolans such as rice husk ash reduced the size of voids in hydrated cement pastes, thus substantially reducing the permeability even at the early ages of 7 to 28 days.

In investigations reported by Gjørv (58) and Markestad (59), water permeability tests were conducted on concretes with cement contents ranging from 100 to 500 kg/m^3. When 10% silica fume was incorporated in the concrete with 100 kg/m^3 cement, the permeability decreased from 1.6x10^{-7} to 4.0x10^{-10} m/s. The permeability of concrete containing 100 kg/m^3 cement and 20% silica fume was found to be equivalent to that of a concrete containing 250 kg/m^3 portland cement without any silica fume. It was also found by Gjorv (58) that the water permeability of concretes containing cement contents of 400 to 500 kg/m^3 was in the range of 10^{-14} to 10^{-15} m/s irrespective of silica fume addition. Thus, in concretes with high cement contents, the effect of silica fume on permeability appears to be negligible.

Skurdal (60) observed in an investigation that curing of concretes, with and without silica fume, at elevated temperatures resulted in increased permeability. The reference concrete and the concrete containing 10% silica fume had 28-day cube compressive strengths of 32 MPa at 20°C curing. The permeability coefficients for these concretes were measured to be 7.2x10^{-13} for reference and 0.5x10^{-13} m/s for silica fume concrete. When cured at a temperature of 30°C, the 28-day strengths of both concretes were slightly lower, and the permeability coefficients were 27x10^{-13} and 0.8x10^{-13} m/s for reference and silica fume concrete, respectively. For 50°C curing, there was a reduction of 20% in the 28-day strengths, and the permeability coefficients were 90×10^{-13} and 74×10^{-13} for reference and silica fume concretes, respectively. In all the different curing conditions, the silica fume concrete had consistently lower permeability values than the reference concrete.

Permeability of concrete to the diffusion of chloride ions is critical in concretes exposed to sea water and de-icing salts. The chloride ions destroy the passivating iron oxide layer over the reinforcement, thus making it vulnerable to corrosion. Therefore, by reducing the mobility of chloride ions into concrete, a measure of protection against corrosion is provided. The incorporation of supplementary cementing materials in concrete, in general, refines the pore structure of the paste, and reduces the penetration of chloride ions into concrete. AASHTO T277-83 "Rapid Determination of Chloride Permeability" test method is the most commonly used method in North America to quantify the chloride-ion penetration into concrete.

Plante and Bilodeau (61) reported that the addition of 8% silica fume significantly reduced the penetration of chloride ion into concrete. With increasing cementitious materials content and decreasing water-to-cementitious material ratio, the chloride ion penetration was reduced further. At a W/C+SF of 0.21 and 500 kg/m^3 cement and 40 kg/m^3 silica fume, the chloride ion penetration was found to be 196 coulombs at 28 days compared to 1246 coulombs for the reference concrete (Fig. 15). This reduction is primarily due to the refined pore structure and increased density of the matrix.

In an investigation by Cohen and Olek (62), measurements of penetration of chloride ions into concretes incorporating various forms of silica fume such as slurry, uncompacted, and densified fume were made. The control concrete was a plain portland cement concrete with 371 kg/m^3 ASTM Type I cement, and 10% of the cement was replaced by silica fume in the silica fume concretes. Water-to-cementitious materials ratio was maintained at 0.35. The 28-day compressive strength of the control concrete was about 43 MPa and that of the silica fume concretes was about 70 MPa. The chloride-ion penetration values measured at 28 days (using the AASHTO T277-83 test method) were 1788 coulombs for the control concrete and 300 to 400 coulombs for the silica fume concretes.

In a study reported by Gautefall (63), the results showed that the chloride content of cement paste exposed to sea water for 6 months was reduced by reducing the water-to-cementitous materials ratio and replacing 5 to 15% of the cement (by mass) by silica fume. In the paste made with a W/C+SF of 0.5 and 15% silica fume replacement, almost no chloride ions were found at depths of 10 mm. In pastes made without silica fume, the chloride content measured was about 1.25% of the weight of the paste.

Carbonation

Silica fume, like other pozzolanic materials, reduces the $Ca(OH)_2$ content of the concrete, and this promotes a faster rate of carbonation. On the other hand, this effect may be offset by the more impermeable nature of silica fume concrete which tends to impede the ingress of CO_2 into concrete. The net effect can be somewhat variable as it depends on various factors such as silica fume content, water-to-cementitious materials ratio and curing conditions, all of which can have a determinant influence on the ultimate $Ca(OH)_2$ content and permeability of the concrete.

Vennesland and Gjorv (64) reported that the use of up to 20% silica fume as an addition to cement in concrete in combination with the use of a plasticizer reduced the rate of carbonation. The concrete specimens were initially moist-cured for a period of 7 days prior to air storage at 60% relative humidity.

Carette and Malhotra (30) compared the rate of carbonation of silica fume concrete with that of reference portland cement concrete up to the age of 3.5 years. The test specimens had been initially moist-cured for 7 days before being stored under ambient room drying conditions. At the water-to-cementitious material ratio of 0.25, they found that all concretes remained free of any noticeable carbonation during the 3.5 year period, whereas at a W/C+SF of 0.40, all specimens exhibited signs of carbonation, the effect being slightly more marked for the silica fume concrete. In general, however, it is agreed that carbonation is not a problem in adequately-cured, high quality low W/C concrete, and this also applies to silica fume concrete.

Chemical Resistance

The chemical attack on concrete causes destructive expansion and decomposition of the cement paste leading to severe deterioration. A permeating solvent as innocuous as water can result in the leaching of calcium hydroxide liberated from the hydration of cement. The ingress of chemicals and acids into concrete react with the calcium hydroxide to form water soluble salts which leach out of concrete, increasing the permeability of concrete, and allowing further ingress of chemicals. Sulphates react with the calcium hydroxide forming ettringite which causes expansion and cracking of the concrete. Silica fume concretes have better chemical resistance than comparable portland cement concretes due to the depletion of calcium hydroxide liberated during the hydration of portland cement by means of pozzolanic reaction with silica fume, thus reducing the amount of lime available for leaching, and due to the decrease in permeability resulting from the refined pore structure of the mortar phase of the concrete.

Sellevold et al. (11) observed that when 20% by weight of cement is replaced by silica fume, the quantity of the calcium hydroxide left in a well hydrated paste is very low. Buil and Acker (54) also showed that in mortar specimens extracted from 28-day old concrete incorporating 25% silica fume by weight replacement of cement, the amount of residual calcium hydroxide remaining was minimal.

The data on sulphate resistance of concrete exposed to 10% sodium sulphate solution were first reported by Bernhardt (65) in Norway. He concluded that the sulphate resistance of concrete improved when 10 to 15% of the portland cement was replaced by silica fume, but, he also noted that the test period was not long enough to draw any firm conclusions.

In a study by Fiska (66), the long-term performance of concrete specimens exposed to the ground water (containing up to 4 g/L of SO_3 with pH varying from 7 to 2.5) in Oslo's alum shale region was monitored by measuring volume changes in the concretes. The concrete specimens,most of which had a W/C of 0.5, were made incorporating various cements and additives. Results of this 20-year old study indicated that the concrete with silica fume at 15% cement replacement and W/C+SF of 0.63 had as much resistance to sulphate attack as a concrete made with a sulphate resisting cement at a W/C of 0.50 (Fig.16). The high sulphate resistance of silica fume concretes are attributed to the following factors:

(a) The refined pore structure, resulting in reduced transport of harmful ions,
(b) Low quantity of calcium hydroxide in the hydrated paste and,
(c) Increase in aluminum incorporated in the hydrates which reduce the amount of alumina available for ettringite formation.

In an investigation by Yamato et al. (67), the chemical resistance of silica fume concretes in solutions of 5% sulphuric acid, 2% hydrochloric acid, and 10% sodium sulphate was studied. The non-air entrained concretes, made incorporating 0, 10, 20, and 30% silica fume and a W/C+SF of 0.55 were water-cured for 91 days before immersion in the solutions. The weight loss and the reduction in dynamic modulus of elasticity were monitored for about one year. Figures 17 and 18 show the weight loss of concretes in these solutions. The authors concluded that the incorporation of silica fume in concrete resulted in significant increases in chemical resistance to the above aggressive solutions. They attributed this to the reduction in permeability and the amount of potentially soluble lime in silica fume concrete.

Durning and Hicks (68) measured the resistance of silica fume concrete to aggressive chemicals. The air-entrained concretes were made incorporating 0, 7.5, 15, and 30% silica fume, with W/C+SF of 0.26 and 0.36. The specimens, 75 x 150-mm in size, were moist cured for 28 days before exposure to 1 and 5% sulphuric acid, 5% formic acid, 5% acetic acid, and 5% phosphoric acid. A mixed sodium and magnesium sulphate solution (0.175 mole/litre of each) was used to evaluate resistance to sulphate attack. Table 5 shows the number of cycles of wetting and drying to 25% mass loss. It was found that as the dosage rate of silica fume is increased, the number of cycles to failure is increased in all solutions. The authors noted that as a 30% silica fume dosage will virtually eliminate all calcium hydroxide in the cement paste, the degradation of concrete with 30% silica fume resulted from the deterioration of the aggregate and of the calcium silicate hydrate paste. In the specimens exposed to the mixed sulphate solutions, no appreciable degradation was observed.

Mehta (69) compared the chemical resistance of low water-to-cement ratio (0.33 to 0.35) concretes exposed to the solutions of 1% hydrochloric acid, 1% sulphuric acid, 1% lactic acid, 5% acetic acid, 5% ammonium sulphate, and 5% sodium sulphate. Specimens of a reference concrete, latex modified concrete, and silica fume concrete with 15% silica fume by weight of cement were used and the criteria for failure was 25% weight loss when fully submerged in the above solutions. The investigation showed that the concrete incorporating silica fume showed better resistance to chemical attacks than the other two types of concretes. The only exception was the silica fume concrete specimens immersed in ammonium sulphate solution which performed poorly. This was attributed to the ability of the ammonium salts to decompose the calcium silicate hydrate in the hydrated cement paste.

However, in another investigation, Popovics et al. (70) observed that silica fume inhibited ammonium sulphate corrosion. Here, mortar specimens made with portland cement, blended cement with 20% slag, blended cements with 15% natural pozzolans, and portland cement with 15% silica fume addition were exposed to 10% ammonium sulphate solution following 28 days of water curing. The authors concluded that sulphate corrosion was predominant in this solution and silica fume was effective in reducing this type of corrosion as well as acid corrosion. This was found to be true in silica fume mortars made with and without superplasticizers, which meant that in mortars with different porosities.

Carlsson et al. (71) reported an investigation in which 100-mm concrete cubes were half immersed in saturated calcium nitrate and ammonium nitrate solutions. The concretes were made with and without a silica fume based admixture which contains 90% silica fume and the rest superplasticizer and portland cement. The results showed that after 108 weeks of exposure to ammonium nitrate solution, the specimens incorporating the silica fume admixture had a weight loss of less than 1%, whereas the specimens without the admixture had weight losses of 15.1%. When comparing the compressive strengths, it was found that specimens made with the admixture had lost less than 20% of the original strength, while the specimens without the admixture had lost more than 70%.

Freezing and Thawing Resistance

Extensive laboratory and field experience in Canada and the U.S.A. has shown that for satisfactory performance of concrete under repeated cycles of freezing and thawing, the cement paste should be protected by incorporating air bubbles, 10 to 100 μm in size, using an air-entraining admixture. Briefly, the most important parameters concerning the entrainment of air in concrete are the air content, bubble spacing factor and specific surface. For satisfactory freezing and thawing resistance, it is recommended that air entrained concrete should have bubble spacing factor ($\bar{L}$) values of less than 200 μm, and specific surface (α) greater than 24 mm^{-1}. Usually fresh portland cement concrete incorporating between 4 to 7% entrained air by volume will yield the above values of $\bar{L}$ and α.

Several investigators have performed studies on the freezing and thawing resistance of silica fume concrete. These include among others Sorensen (72), Gjorv (58), Carette and Malhotra (14), Malhotra (37), Yamato et al. (73), Hooton (74), Hammer and Sellevold (75), Virtanen (76), and Pigeon et al. (77).

In studies at CANMET, the freezing and thawing resistance of concrete incorporating silica fume was investigated using ASTM Standard C 666. The method is designated as rapid because it allows for alternatively lowering the temperature of specimens from 4.4 to -17.8°C and raising it from -17.8 to 4.4°C in not less than 2 nor more than 4 hr. The customary accepted duration of testing is 300 cycles, which can be completed in 25 to 50 days. In Procedure A of ASTM C 666, both freezing and thawing occur with the specimens surrounded by water, while in Procedure B of the test the specimens freeze in air and thaw in water. Procedure B is less severe than Procedure A. The requirements for Procedure A are met by confining the specimen and surrounding water in a suitable container. The specimens used normally are prisms not less than 76 mm nor more than 137 mm in width and depth, and between 356 and 406 mm in length. The mixture proportions, the properties of fresh concrete, and the freezing and thawing test results for one such study are shown in Tables 6 to 9. The air-entrained concrete incorporated 5 to 30% condensed silica fume by weight as replacement for portland cement. The W/(C+SF) was maintained at 0.40, and any loss in slump due to the use of silica fume was compensated for by the use of a naphthalene-based superplasticizer. The test data revealed satisfactory performance of air-entrained concrete except for those concretes which contained 20 and 30% silica fume (Table 8, Fig. 19 and 20). The excessive expansions and low value of the relative dynamic modulus shown in Fig. 19 and 20 occurred in spite of the fact that the $\bar{L}$ and α

values (Table 9) obtained were satisfactory. The authors of the investigation had speculated that the poor performance of the concretes in question was due to the high amount of condensed silica fume used in concrete, resulting in a very dense cement matrix that, in turn, might have adversely affected the movement of water (14).

In another CANMET investigation, the freezing and thawing resistance of non air-entrained and air-entrained concrete incorporating various percentages of silica fume was compared (37). Once again, the air-entrained concrete incorporating 30% silica fume failed to meet the durability criterion (Fig. 21), though in this case the poor performance of the concrete was attributed to the unsatisfactory $\bar{L}$ values. The above study led to the following conclusions:

> "Non air-entrained concrete: Non air-entrained concrete, regardless of the W/(C+SF), and irrespective of the amountof condensed ilica fume shows very low durability factors and excessive expansion when tested in accordance with ASTM C666 (Procedure A or B). The concrete appears to show somewhat increasing distress with increasing amounts of the fume. Therefore, the use of non air-entrained condensed silica fume concrete is not recommended when it is to be subjected to repeated cycles of freezing and thawing.
>
> Air-entrained concrete: Air-entrained concrete, regardless of the W/(C+SF) and containing up to 15% condensed silica fume as partial replacement for cement, performs satisfactorily when tested in accordance with ASTM C 666 Procedures A and B. However, concrete incorporating 30% of the fume and a W/(C+SF) of 0.42, performs very poorly (durability factors less than 10) irrespective of the procedure used. This is probably due to the hardened concrete having high values of $\bar{L}$ or due to the high amount of condensed silica fume in concrete, resulting in a very dense cement matrix system that, in turn, might have adversely affected the movement of water. It was difficult to entrain more than 5% air in the above type of concrete and this amount of air may or may not provide satisfactory $\bar{L}$ values in hardened concrete for durability purposes. The users are therefore asked to exercise caution when using high percentages of condensed silica fume as replacement for portland cement in concretes with W/(C+SF) of the order of 0.40, if these concretes are to be subjected to repeated cycles of freezing and thawing."

One recent CANMET study reported results of an investigation dealing with freezing and thawing resistance of concrete in which non air-entrained concrete was proportioned to have a W/(C+SF) ranging from 0.35 to 0.25 (78). One series of the mixtures incorporated 10 and 20% silica fume (Table 10). Large quantities of a superplasticizer were added to obtain slumps in the order of 200 mm. The non air-entrained concrete test specimens, with and without the fume, when exposed to rapid freezing and thawing tests (ASTM C 666 Procedure A) started showing distress at less than 30 cycles, and developed major cracks at 50 cycles. No microcracks or scaling was observed in any instance (Fig. 22). The visual observations of the width of the cracks indicated that the prisms without the fume had performed marginally better. The failure of the very low W/(C+SF) concretes in the above freezing and thawing test is probably due to the availability of freezable water in the pores of the high strength, very low permeability matrix having unsatisfactory strain capacity.

Data by Yamato et al. (73), Hammer and Sellevold (75), and Virtanen (76) are, in general, in agreement with the CANMET's data. Hammer and Sellevold have tried to explain the poor performance of non air-entrained silica fume concretes (5 to 10% silica fume) with a water-to-cementitious ratio of 0.28 to 0.37 in ASTM C 666 as follows:

> "There appears to be a conflict between the calorimeter data that little or no ice forms in the strongest concretes and the fact that ASTM C 666 tests showed poor performance of the same type of concrete. We suggest that thermal incompatibility between aggregate and binder may cause this, a possibility that should be investigated further".

Frost Resistance in the Presence of De-Icing Salts

Limited published data are available on the de-icing salt scaling resistance of silica fume concrete. Gagne et al. (78) have reported that the frost resistance of concrete incorporating 6% silica fume as an addition to cement is satisfactory in the procedure of de-icing salt scaling tests (ASTM C 672). This was true for both air-entrained and non air-entrained concrete. The W/(C+SF) of the concretes ranged from 0.23 to 0.30 and the concretes were air dried for 28 days before being subjected to the de-icing salt scaling test.

Bilodeau and Carette (80) investigated the de-icing salt scaling resistance of portland cement concrete and concretes incorporating 8% silica fume as replacement for cement using ASTM C 672 test. In their investigation, superplasticized and non-superplasticized concretes were made, with water-to-cementitious materials ratio ranging from 0.40 to 0.65, all having adequate air-void parameters. They reported that, in general, all concretes performed satisfactorily under the action of de-icing salts, though the silica fume concrete exhibited a slightly inferior performance when compared to that of portland cement concrete. In particular, silica fume concrete with a W/(C+SF) of 0.60 showed appreciable weight loss after 50 cycles.

De-icing salt scaling studies by Hammer and Sellevold (75) showed that both normal density and lightweight aggregate high strength concretes with W/(C+SF) of lower than or equal to 0.37 and 0.47, respectively, had acceptable resistance to freezing and thawing exposure in saline water even without air-entrainment. The tests were conducted in accordance with the Swedish Standard SS 13 72 44 which is somewhat similar to ASTM C 672. However, when 5 to 10% of portland cement was replaced by silica fume, the scaling resistance was reduced. No explanation for this inferior performance of silica fume concrete was offered.

Alkali-Aggregate Reactivity

In 1940 it was reported by Stanton (81) that chemical reactions involving aggregates in concrete can result in abnormal expansion and cracking of concrete. This reaction is known as alkali-silica reaction, and involves a reaction between alkalis (Na_2O and K_2O) from the cement or from other sources, with hydroxyl and certain siliceous constituents that may be present in the aggregates. The principal reaction products are gelatinous hydrates which expand as water is imbibed, and exert pressure on the surrounding mortar.

Certain carbonate rocks are also prone to attack by alkalis, resulting in detrimental expansion and cracking of concrete. This detrimental reaction is usually associated with argillaceous dolomitic limestone, and is designated as the expansive alkali-carbonate reaction. According to Swenson and Gillott (82), the reaction involves destruction of dolomitic crystals by the alkaline solutions and the release of entrapped active clay minerals that absorb water directly and swell. The principal reaction products include calcite and films of Ca-Mg hydroxides and silicates.

One of the most effective methods of controlling the alkali-aggregate reactions in concrete is to use low-alkali cements. However this is not always possible or economical. In such situations mineral admixtures i.e., natural pozzolans, fly ash, and granulated blast-furnace slags have been used successfully to control the expansion associated with the alkali-silica reaction. A number of laboratory investigations have indicated that silica fume like other pozzolans is effective in controlling the above-mentioned expansions due to alkali-silica reactions. However, the percentage of silica fume to be incorporated into concrete would depend upon the type of reactive aggregate, the exposure conditions, the alkali and silica contents, the silica fume used, the type of cement used and the water-to-cementitious materials ratio of the mixture. Published data indicate that the percentage replacement of cement by silica fume may range from about 10 to 15% (Figure 23) (83). CANMET-funded studies have indicated that silica fume is not effective in controlling expansions due to the alkali-carbonate reactions (Figure 24) (84).

Hypotheses concerning the mechanisms by which the use of silica fume may contribute to the control of alkali-silica reactions are many. These may be summarized as follows:

(a) Alkali dilution by the partial replacement of portland cement by silica fume
(b) Pozzolanic reaction and the formation of C-S-H with low CaO/SiO_2 ratio which has increased capacity to accommodate Na_2O and K_2O in its structure thereby reducing the hydroxyl ion concentration
(c) Reduced permeability with resulting lower chloride-ion mobility
(d) Dense microstructure.

At present no significant well documented data are available as to the long-term effectiveness of silica fume in controlling alkali-silica expansions in actual field structures. Gudmundsson and Asgeirsson (85) has reported a case history in Iceland, and Aitcin and Regourd (86) have reported a case history in Quebec where silica fume has been used to control potential expansions due to alkali-silica reactions in concrete. Based upon the current state of knowledge it is recommended that those contemplating the use of silica fume to control expansion due to alkali-silica reactions should perform accelerated tests in the laboratory using the materials to be employed on a job site to determine the percentage of silica fume to be used as replacement for cement.

Resistance to Elevated Temperatures

Effects of high temperatures on the behaviour of silica fume concrete are essentially similar to those observed with portland cement concrete. Following early reported cases of disintegration of silica fume concrete at temperatures in the range of 300 to 400°C (87), there had been suspicion that silica fume might contributed to a reduction in durability of resistance concrete when exposed to high temperatures. However, these isolated cases included concretes made with exceptionally low W/C+SF, and it was suggested later that such observations were not unusual for ultra high strength and impermeable concrete because of the building of high vapour pressure inside the concrete specimens under heating.

A study by Shirley et al. (88) has shown that the fire resistance of silica fume concrete of normal structural quality is in fact as good as that of normal portland cement concrete. From his review of various investigations on the fire resistance of high strength concrete, Jahren (89) concluded that there was little or no evidence that concrete materials components such as silica fume has a negative effect on the fire resistance of high strength concrete, the important factors being the physical characteristics of the system after hardening and the conditions under which it is used.

Abrasion Resistance

Properties of concrete known to have a large influence on its resistance to abrasion include, among others, compressive strength, and bond strength between the cement paste and aggregate. As discussed earlier, the use of silica fume is generally associated with improvement in both of the above properties, and this largely explains why silica fume concrete has been observed to exhibit better abrasion resistance than comparable portland cement concrete (90). The particularly good abrasion-erosion resistance of silica fume concrete in underwater applications has been reported by several investigators (38,91).

SPECIFICATIONS FOR SILICA FUME AS A MATERIAL

Even though the silica fume has been in use for the past 10 years, the only comprehensive specification for the material is that one issued by the Canadian Standards Association (92). ASTM has a draft specification, and this should be ready for publication in 1994. A Norwegian specification on the subject is also under preparation, and a draft has been circulated for comments.

Canadian Specification for Silica Fume

The highlights of the Canadian specification CSA-A23.5-M86 are as follows:

A. Chemical requirements

The chemical requirements covering SiO_2, SO_3, and loss on ignition (LOI) as approved by the Canadian Standard are shown in Table 11 together with the limits for the natural pozzolans and fly ashes. According to Isabelle (93), the rationale for the selection of limits shown in Table 11 is as follows.

> "The most important of these requirements is the minimum SiO_2 content which has been placed at 85% because it has been found that silica fume having at least 85% SiO_2 generally performs well whereas that having a lesser amount of SiO_2 does not perform as well. Furthermore, the silica fumes from Canadian sources have SiO_2 content well in excess of the specified minimum.

Silica fume contains very little SO_3, principally because of the low sulphur carbon used in the manufacturing process. This is done on purpose in order to obtain silicon and ferro-silicon alloys of the highest quality. In view of the above, the Subcommittee debated the need to put any limit at all on SO_3 but finally allowed for a maximum of 1%.

Loss of ignition has been set at 6% maximum although in practice it is generally much lower. However, when wood chips are used in the manufacture of silicon or ferro-silicon alloys, some of the fine wood chips may find their way into the fume causing an increase in the loss on ignition; they do not adversely affect its intrinsic quality."

B. Physical requirements

As in the existing specifications for fly ash, the physical requirements have been divided into two separate parts, one covering the mandatory and the other covering the optional requirements.

The mandatory requirements covering accelerated pozzolanic activity index, fineness, soundness, and uniformity are shown in Table 12. The non-mandatory or optional requirements governing drying shrinkage, air entraining admixture dosage, and reactivity with cement alkalis are given in Table 13.

The inclusion of the accelerated pozzolanic activity index requirement is premature because of the lack of sufficient published data on the test method. The fineness measurement by determining the percentage retained on a 45 μm sieve is also arbitrary. The correct way to determine the fineness of silica fume is by the BET nitrogen adsorption technique; the standard should have included this technique. The uniformity and density requirements are the same as those for natural pozzolans and fly ashes in the existing standard.

The maximum allowable autoclave expansions have been set at 0.20%. The rationale for the selection of a value of 0.20% is based on the premise that the limited amount of data available to the Subcommittee indicated values much lower than the one being adopted.

As for the non-mandatory physical requirements, the drying shrinkage and uniformity of the addition rate of an air-entraining admixture are the same as those for natural pozzolans. The requirement that condensed silica fume must reduce mortar expansion due to the reaction between alkalis in cement and reactive silica in aggregates at 14 days by at least 80% as compared with control is somewhat arbitrary because no detailed published data are yet available. The probable rationale is that because silica fume contains a high percentage of amorphous silica, it should be more effective in controlling the alkali-silica reaction in concrete. Some evidence suggests that cement paste containing silica fume adsorbs more alkali and alkaline earth cations than the control cement containing no silica fume.

C. Methods of Test

The testing methods for ensuring that condensed silica fume meets the specification requirements are given in Table 14. These are primarily ASTM test methods except for the determination of SO_3 and the percentage retained on a 45 μm sieve for which reference is made to test methods given in Canadian Standard CAN3-A5.

APPLICATIONS OF SILICA FUME CONCRETE

There are a number of applications in which silica fume has been used successfully. These include the following.

(i) **To produce concrete with very low permeability to control ingress of chloride ions.** Typical applications are bridge-deck overlays, repair materials for parking structures and concrete for marine applications.

(ii) **To produce high strength concrete with strengths exceeding 100 MPa.** Typical applications include concrete for columns for very tall structures.

(iii) **To produce high-strength lightweight concrete with strengths exceeding 50 MPa.** Typical applications include offshore drilling platforms in the Beaufort sea.

(iv) **To save cement.** Typical applications include ready-mixed concrete containing about 8% silica fume with corresponding reduction in cement content.

(v) **To reduce chemical attack in industrial plants.** Typical applications include floors and silos in fertilizer plants, food in chemical industry and floor processing plants.

(vi) **To produce shotcrete with reduced rebound loss, to increase resistance to washout by wave action and to increase thickness of shotcrete layers.** Typical applications include slope stabilization, rock support system in tunnels, and repair of marine structures.

(vii) **To achieve high strength at early ages.** Typical applications include precast concrete elements.

(viii) **To improve early-age strength of concrete made with fly ash or slag.** Typical possible application should include structural high-strength concrete incorporating fly ash or slag.

(ix) **To control alkali-silica reaction in concrete.** Well documented case histories are lacking.

(x) **To improve sulphate resistance of concrete.** Well documented case histories are not yet available.

CONCLUDING REMARKS

Silica fume, a highly amorphous material, is a relatively new pozzolanic admixture, and is rapidly gaining acceptance for use in concrete. The principal areas of application of silica fume concrete include low W/C+SF high strength concretes to resist aggressive chemicals, and concretes for bridge decks and parking structures. Long-term data on the performance of silica fume as a mineral admixture to control alkali-aggregate reaction are lacking, and caution is recommended in the selection of the percentage of silica fume to be used.

The use of superplasticizer is almost mandatory in the proper use of silica fume in concrete. It is emphasized that good curing immediately after concrete placement is essential for avoiding cracks, and for satisfactory strength development. In spite of the very low permeability of silica fume concrete, air-entrainment is essential when the concrete is to be exposed to repeated freezing and thawing conditions.

Progress is being made in North America and Europe in the development of specifications for the material and its proper use in concrete.

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TABLE 1 - Physical Characteristics of Silica Fumes*

Physical Characteristics	Nebesar & Carette results (24 samples)		Pistilli et al. results	
	Si-SF	FeSi-75%-SF	Mixture of Si & FeSi-75%-SF (32 samples)	FeSi-75%-SF (30 samples)
Fineness by 45 μm sieve (% passing)	94.6	98.2	94.0	96.3
Specific surface area (m^2/kg)	20,000^A	17,200^A	3,750^B	5,520^B
Pozzolanic activity index with portland cement (%)	102.8	96.5	91.9	95.3
Water requirement (%)	138.8	139.2	140.1	144.5
Pozzolanic activity index with lime (MPa)	8.9	-	7.0	9.1
Specific gravity	-	-	2.27	2.26

^{A} Nitrogen adsorption.

^{B} Blaine permeability.

* From reference 1.

TABLE 2 - Chemical Characteristics of Silica Fumes*

Chemical Composition	Nebesar & Carette results (24 samples)		Pistilli et al. results	
	Si-SF	FeSi-75%-SF	Mixture of Si-SF & FeSi-75%-SF (32 samples)	FeSi-75% (6 samples)
SiO_2	93.7^A	93.2	92.1	91.4
Al_2O_3	0.28	0.3	0.25	0.57
Fe_2O_3	0.58	1.1	0.79	3.86
CaO	0.27	0.44	0.38	0.73
MgO	0.25	1.08	0.35	0.44
Na_2O	0.02	0.10	0.17	0.20
K_2O	0.49	1.37	0.96	1.06
S	0.20	0.22	-	
SO_3	-	-	0.36	0.36^B
C	3.62	1.92	-	
LOI	4.4	3.1	3.20	2.62^B

^{A} Calculated.

^{B} From 30 samples.

* From reference 1.

TABLE 3 - Compressive Strength and Modulus of Elasticity of Concrete with Different W/(C + BFS)*

W/(C+BFS)[A]	Mix No.	Type of Mixture[B]	Compressive strength of 100 x 200-mm cylinders (MPa)		Modulus of elasticity (GPa) 28 days
			7-day	28-day	
0.40	1	Reference	38.3	47.1	36.0
	2	Control	25.7	46.7	36.3
	3	5% Silica fume	30.3	57.1	36.1
	4	10%	34.3	59.3	36.9
	5	15%	34.5	60.7	35.8
	6	20%	37.6	60.1	34.9
0.50	7	Reference	30.4	36.5	33.3
	8	Control	18.8	33.0	34.1
	9	5% Silica fume	20.7	38.9	33.6
	10	10%	25.1	45.5	34.4
	11	15%	28.5	48.8	34.0
	12	20%	29.3	47.5	33.8
0.65	13	Reference	21.0	28.3	31.3
	14	Control	10.0	20.6	32.6
	15	5% Silica fume	11.9	26.2	31.4
	16	10%	15.7	31.7	32.8
	17	15%	18.5	35.3	31.8
	18	20%	22.7	38.0	31.3

Note: Each value is average of three tests.

[A] Water/(cement + blast furnace slag) by weight = W/(C + BFS).

[B] Reference mixture: 100% normal Portland cement; control mixture: 50% normal Portland cement plus 50% BFS; silica fume mixture: 50% normal Portland cement plus 50% BFS plus additions of silica fume.

* From reference 52.

TABLE 4 - Summary of Creep Test Results*

Mix No.	$\frac{W}{(C+SCM)}$	Silica Fume, %	Fly Ash, %	Slag, %	Age at Loading, Days	Applied Stress, MPa	Duration of Loading, Days	Initial Elastic Strain, x 10^{-6}	Creep^{+} Strain, x 10^{-6}
1	0.22	0	0	0	35	28.0	385	717	1505
2	0.22	7	0	0	35	28.0	380	677	713
3	0.22	12	0	0	35	28.0	378	694	836
4	0.22	0	25	0	36	28.0	372	750	1069
5	0.22	7	25	0	35	28.0	371	645	749
6	0.22	0	0	40	35	28.0	366	690	1005
7	0.22	7	0	40	35	28.0	364	632	641

[+] Creep strain = total load induced strain - initial elastic strain.

* From reference 53.

TABLE 5 - Resistance of Concrete Containing Silica Fume to Chemical Attack[*]

Sample Code	Silica Fume, %	Environment	Cycles to 25% mass loss	Percent Loss at 25 cycles
1SA	0	1% sulfuric acid	13	-
2SA	7.5	1% sulfuric acid	19	-
3SA	15	1% sulfuric acid	22	-
4SA	30	1% sulfuric acid	32	22
1AA	0	5% acetic acid	8	-
2AA	7.5	5% acetic acid	17	-
3AA	15	5% acetic acid	32	22
4AA	30	5% acetic acid	>60	8
1FA	0	5% formic acid	17	-
2FA	7.5	5% formic acid	32	-
3FA	15	5% formic acid	>50	12[†]
4FA	30	5% formic acid	>50	8[†]
1SA5	0	5% sulfuric acid	4	-
2SA5	12.5	5% sulfuric acid	5	-
3SA5	25	5% sulfuric acid	5.3	-
1PA	0	5% phosphoric acid	12	-
2PA	12.5	5% phosphoric acid	13.6	-
3PA	25	5% phosphoric acid	17.3	-

[†] 30 cycles.

[*] From reference 68.

TABLE 6 - Mix Proportions of Concrete for Freezing and Thawing Tests*

Mix	Replacement of cement by silica fume (%)	W/(C+SF)[A]	A/(C+SF)[B]	Quantities (kg/m^3)		AEA (mL/m^3)[C]	SP[D], % by wt of (C+SF)
				Cement	Silica fume		
1	0	0.40	4.43	400	0	170	0.0
2	5	0.40	4.42	381	20	190	0.1
3	10	0.40	4.40	367	41	240	0.8
4	15	0.40	4.38	342	61	540	1.0
5	20	0.40	4.37	322	81	770	1.9
6	30	0.40	4.33	285	122	1090	2.7

[A] Water/(cement + silica fume) (by weight).
[B] Aggregate/(cement + silica fume) (by weight).
[C] Air-entraining admixture.
[D] Superplasticizer.
* From reference 14.

TABLE 7 - Properties of Fresh Concrete in Freezing and Thawing Tests*

Mix	Type	W/(C+SF)[A]	Properties of Fresh Concrete			
			Temp. (°C)	Slump (mm)	Unit wt (kg/m^3)	Air content (%)
1	Control	0.40	21	75	2340	5.1
2	5% SF+SP[B]	0.40	23	65	2330	4.4
3	10% SF+SP	0.40	22	60	2370	3.8
4	15% SF+SP	0.40	22	75	2330	4.5
5	20% SF+SP	0.40	24	180[C]	2330	4.6
6	30% SF+SP	0.40	23	160[C]	2340	4.2

[A] Water/(cement + silica fume)(by weight).
[B] SP = superplasticizer.
[C] Increased slumps are caused by the increased dosage of the superplasticizer.
* From reference 14.

TABLE 8 - Summary of Test Results after Freezing and Thawing[*]

Mix	Type	W/(C+SF)[A]	At the Beginning of Freezing and Thawing Cycles			At the End of Freezing and Thawing Cycles						
			Weight (kg)	Longitudinal Resonant Frequency (Hz)	Pulse Velocity (m/sec)	No.of Cycles	Weight (kg)	Longitudinal Resonant Frequency (Hz)	Pulse Velocity (m/sec)	Length Change (%)[B]	Relative Dynamic Modulus (%)[C]	Residual Flexural Strength (%)[C]
1	Control	0.40	7.402	5175	4660	600	7.289	5200	4780	0.028	101	74
2	5% SF+SP	0.40	7.335	5200	4670	500	7.286	5200	4730	0.018	100	75
3	10% SF+SP	0.40	7.350	5250	4720	500	7.329	4900	4610	0.059	87	66
4	15% SF+SP	0.40	7.327	5125	4540	525	7.289	5000	4530	0.068	95	78
5	20% SF+SP	0.40	7.285	5150	4590	425	7.251	4050	3810	0.206	62	49
6	30% SF+SP	0.40	7.224	5150	4660	300	7.205	4250	3820	0.170	68	32

[A] Water/(cement + silica fume) (by weight).

[B] Gaugelength = 358 mm.

[C] At the end of respective cycling, residual strength was determined in relation to reference moist-cured specimens of same age.

[*] From reference 14.

TABLE 9 - Air-void Characteristics of Hardened Concrete for Freezing and Thawing Tests*

Mix	Type	Air Content (%)[A]	Air/paste Ratio	Specific Surface (mm^{-1})	Spacing Factor $\bar{L}$(μm)
1	Control	4.5 (5.1)	0.176	22.3	221
2	5% SF+SP	4.4 (4.4)	0.166	29.4	172
3	10% SF+SP	3.6 (3.8)	0.135	16.3	340
4	15% SF+SP	4.3 (4.5)	0.163	19.0	268
5	20% SF+SP	4.6 (4.6)	0.178	17.1	285
6	30% SF+SP	4.4 (4.2)	0.172	17.3	288

[A] Values in parentheses refer to air content of fresh concrete.

* From reference 14.

TABLE 10 - Mix Proportions and Properties of Fresh and Hardened High-Strength Concrete*

Mix No.	W/(C+SF)	A/(C+SF)	Cement (kg/m^3)	Silica Fume		Super Plasticizer (kg/m^3)	Properties of Fresh Concrete			Compressive strength at 28 days (MPa)	Freezing and Thawing Test Results	
				%	kg/m^3		Slump (mm)	Air Content (%)	Density (kg/m^3)		No. of Cycles Completed	Durability Factor
1	0.35	5.2	377	0	0	4.8	165	2.0	2460	51.5	66	6
2	0.35	5.1	342	10	38	6.2	140	1.8	2460	61.4	70	6
3	0.35	5.1	306	20	77	9.4	165	1.2	2470	68.5	70	10
7	0.30	4.3	443	0	0	7.6	215	1.9	2480	61.9	67	12
8	0.30	4.3	400	10	44	9.0	195	1.3	2475	79.3	67	3
9	0.30	4.2	356	20	89	12.5	230	1.1	2460	80.0	70	3
13	0.25	3.4	531	0	0	10.4	230	2.0	2490	65.9	89	11
14	0.25	3.4	482	10	54	11.9	230	1.3	2490	81.7	89	5
15	0.25	3.4	428	20	107	14.2	215	1.5	2465	87.1	89	8

Note: All ratios are by weight; compressive strength was determined on 150 x 300-mm cylinders; freezing and thawing test used was Procedure A of ASTM C666; cement type - ASTM Type 1, coarse aggregate - crushed limestone, fine aggregate - natural sand, superplasticizer - a naphthalene-based product; freezing and thawing test results indicate complete failure of test prisms at less than 100 cycles.

* From reference 78.

TABLE 11 - Chemical Requirements*

Property	Material Type			
	N	F	C	U (Silica fume)
SiO_2, min (%)	-	-	-	85
SO_3, max (%)^A	3.0	5.0	5.0	1.0
Loss on ignition max (%)	10.0	12.0	6.0	6.0

Note: N refers to natural pozzolans, F, low-calcium fly ashes, and C, high-calcium fly ashes.

A This limit may be exceeded, provided that the supplementary cementing material, when tested in combination with the particular portland cement with which it is to be used, exhibits expansion not in excess of 0.020% at 14 days when tested in accordance with Clause 7.5.5 of CAN3-A5. In the test mix, replace a mass of portland cement by an equal mass of supplementary cementing material in the amount of 20% generally or for Type U in the amount of 10% or the anticipated maximum field replacement percentage, whichever is greater.

* From reference 93.

TABLE 12 - Mandatory Physical Requirements*

Property	Material Type			
	N	F	C	U (silica fume)
Accelerated pozzolanic activity index, min, % of control	68	68	68	85
Fineness, max, % retained on 45 μm sieve (wet sieving)	34	34	34	10
Uniformity requirements: soundness - autoclave expansion or contraction (%)	0.8	0.8	0.8	0.2
Relative density, max variation from average (%)	5	5	5	5
Fineness, max variation from average, % points	5	5	5	5

Note: The accelerated pozzolanic activity index determination is performed by carrying out the test at 65°C instead of 38°C as for the normal pozzolanic index determination.

* From reference 93.

TABLE 13 - Optional Physical Requirements for Pozzolans*

Property	Material Type			
	N	F	C	U (Silica fume)
Increase of drying shrinkage, max % points of control	0.03	0.03	0.03	0.03
Uniformity of addition rate of an air-entraining agent: max variation from average (%)	20	20	20	20
Reactivity with cement alkalis: min reduction (%)	75	60	60	80

* From reference 93.

TABLE 14 - Methods of Test for Pozzolans*

	Reference Standards
Chemical tests	
Sulphur trioxide (SO_3)	CAN3-A5
Loss on ignition (LOI)	ASTM C311
Silica (SiO_2)	ASTM C114
Physical tests	
Pozzolanic activity index, 7 days	ASTM C311 (Sect. 29 to 32) except store both control and test cubes 6 days at 65°C and test at 7 days
Drying shrinkage	ASTM C311 (Sect. 22 to 32) except that for testing silica fume, use 500 g of portland cement, 50 g of silica fume, and 1325 g of sand in the test mix
Relative density	ASTM C311 (Sect. 20)
Soundness	ASTM C311 (Sect. 25)
Reactivity with cement alkalis	ASTM C441
Uniformity of air content	ASTM C311 (Sect. 27 and 28)
Fineness: wet sieved on 45 µm sieve	CAN3-A5

* From reference 93.

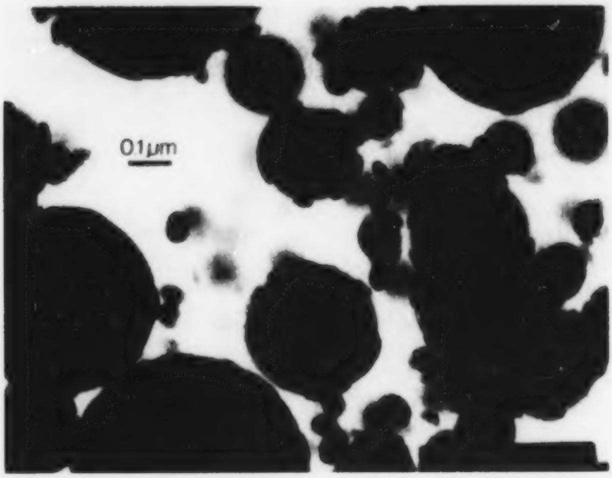


Fig. 1 - Transmission electron micrograph of silica fume particles. From reference 6.

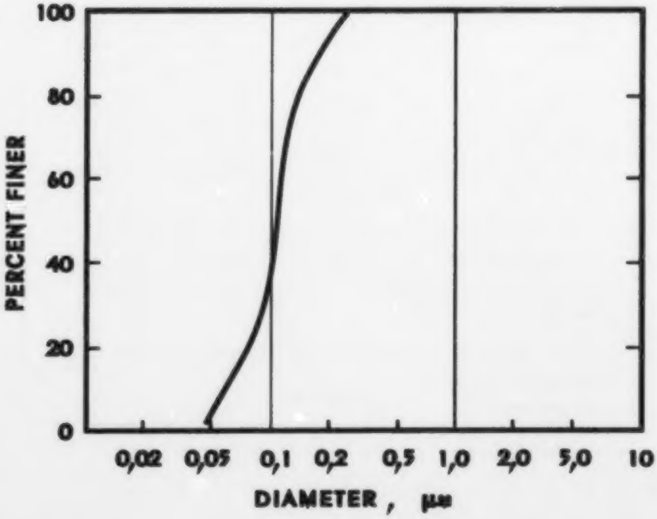


Fig. 2 - Typical particle size distribution of silica fume. From reference 6.

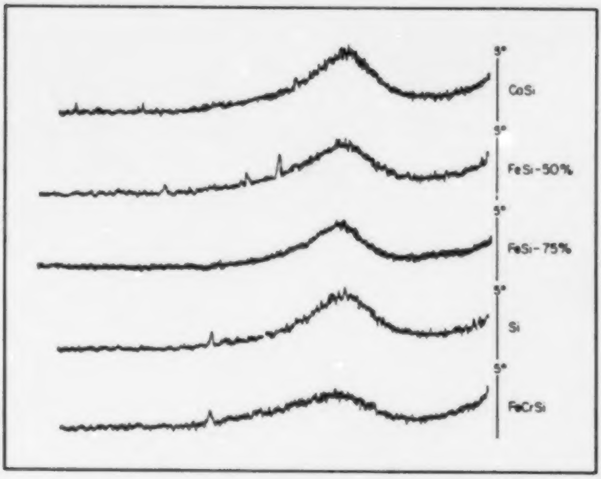


Fig. 3 - X-ray diffractograms of different types of silica fume. From reference 6.

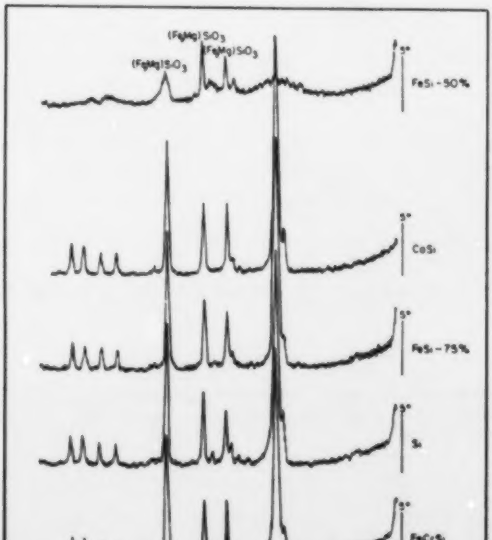


Fig. 4 - X-ray diffractograms of different types of silica fume after heating at 1100°C. From reference 6.

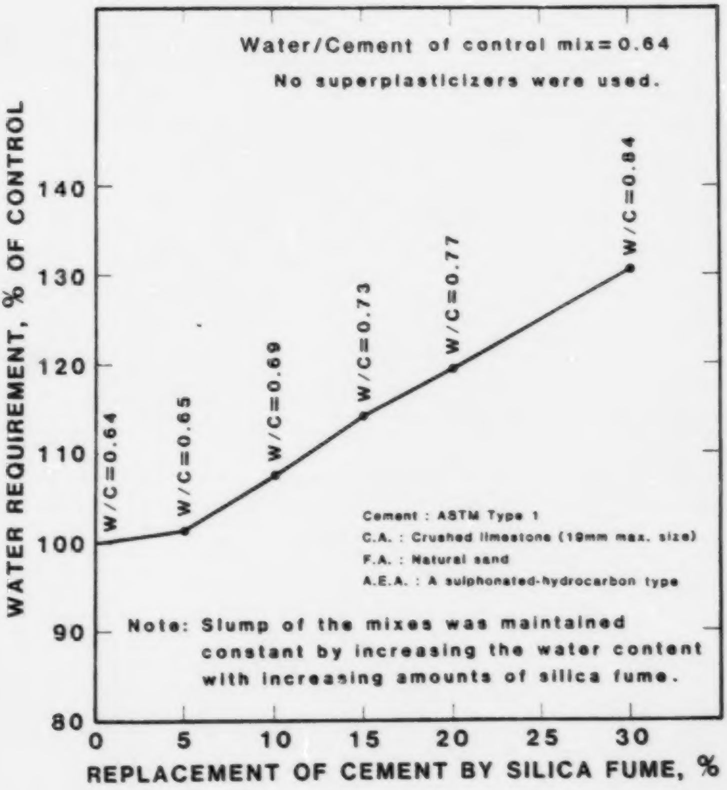


Fig. 5 - Relation between water requirement and dosage of silica fume for concrete with a W/(C+SF) of 0.64. From reference 14.

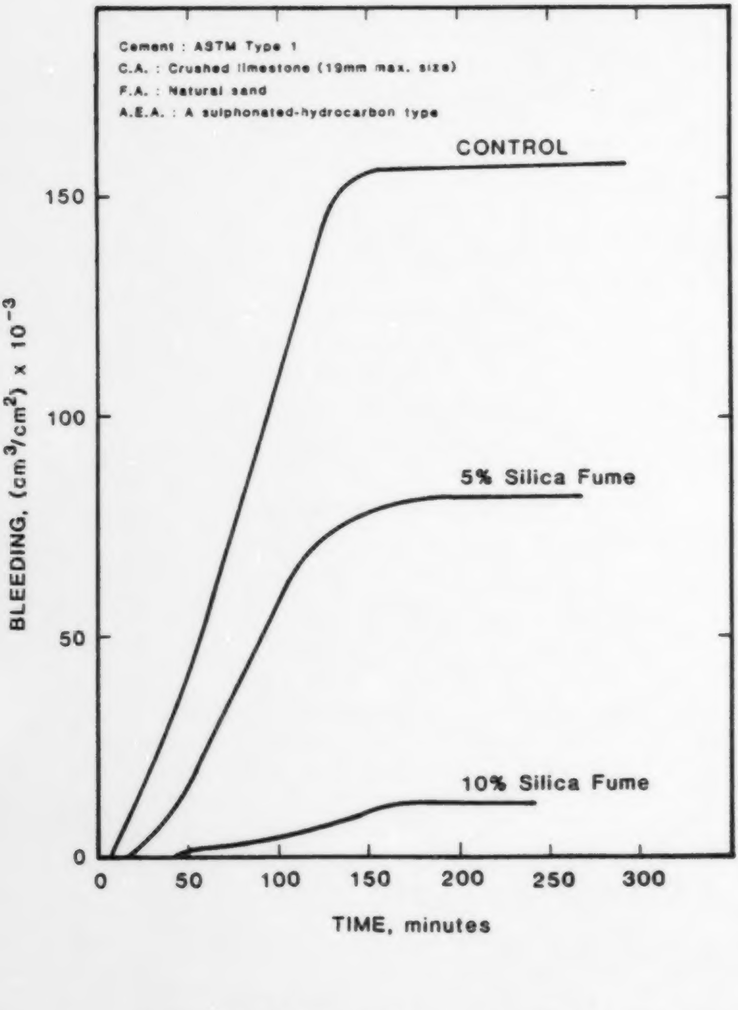


Fig. 6 - Bleeding rate of control and silica fume concrete. From reference 19.

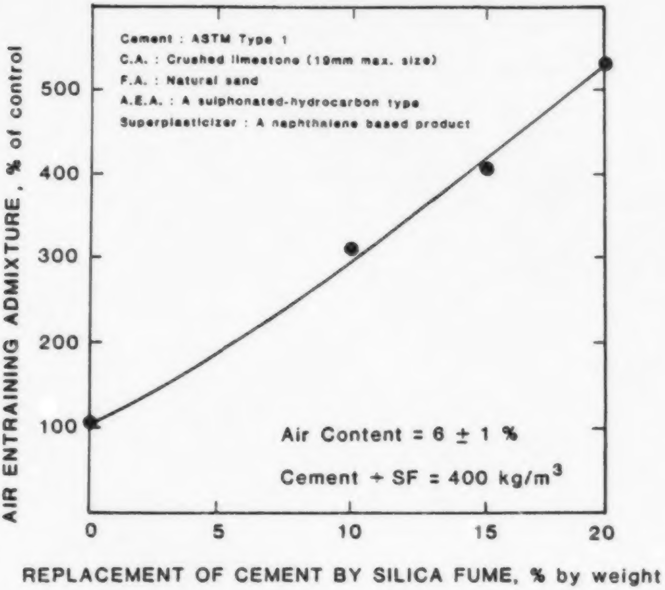


Fig. 7 - Relation between air entraining admixture demand and dosage of silica fume. From reference 14.

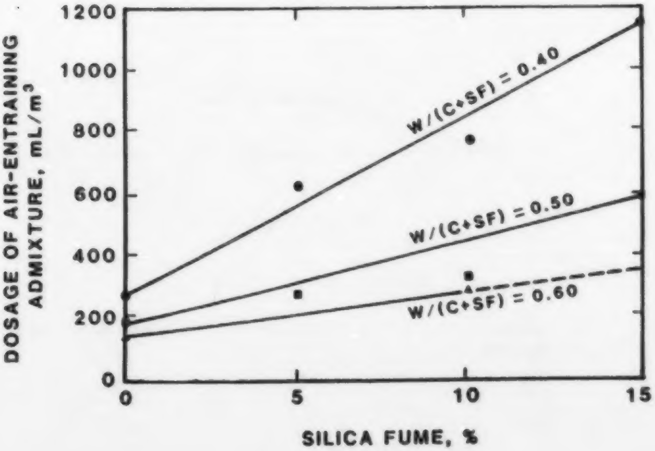


Fig. 8 - Demand of air-entraining agent vs content of silica fume. From reference 22.

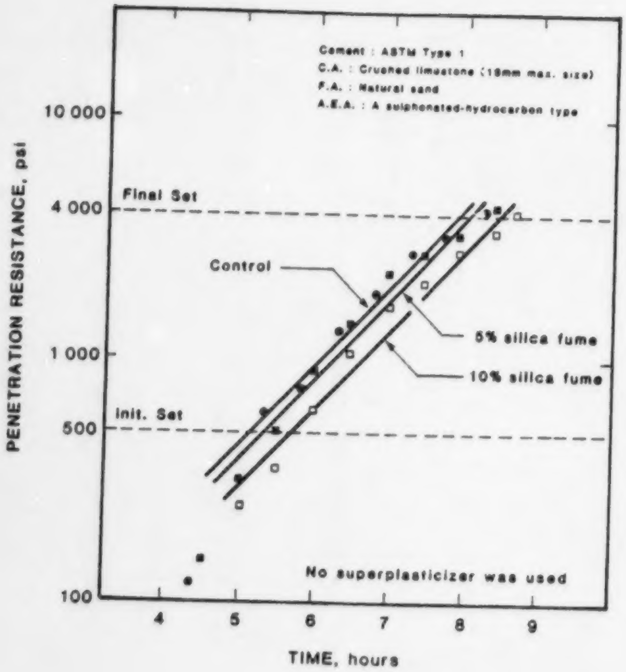


Fig. 9 - Penetration resistance of control and nonsuperplasticized silica fume concrete. From reference 19.

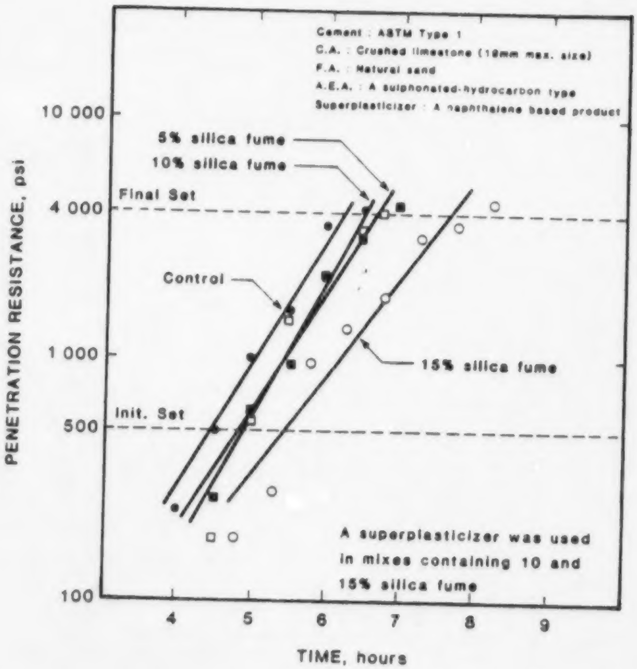


Fig. 10 - Penetration resistance of control and superplasticized silica fume concrete. From reference 19.

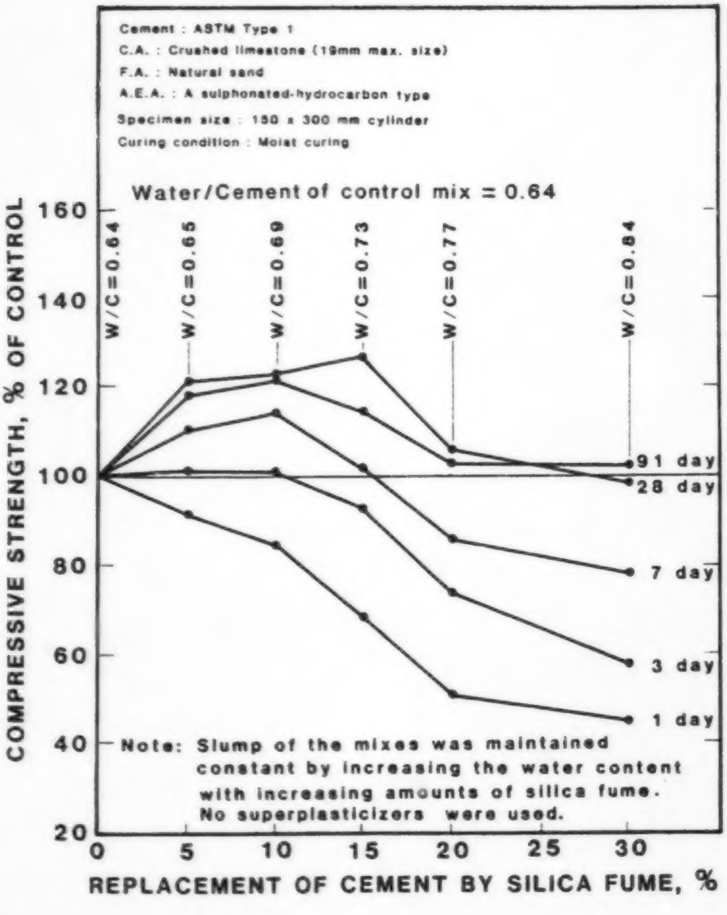


Fig. 11 - Relation between compressive strength of concrete and dosage of silica fume, (W/C = 0.64). From reference 14.

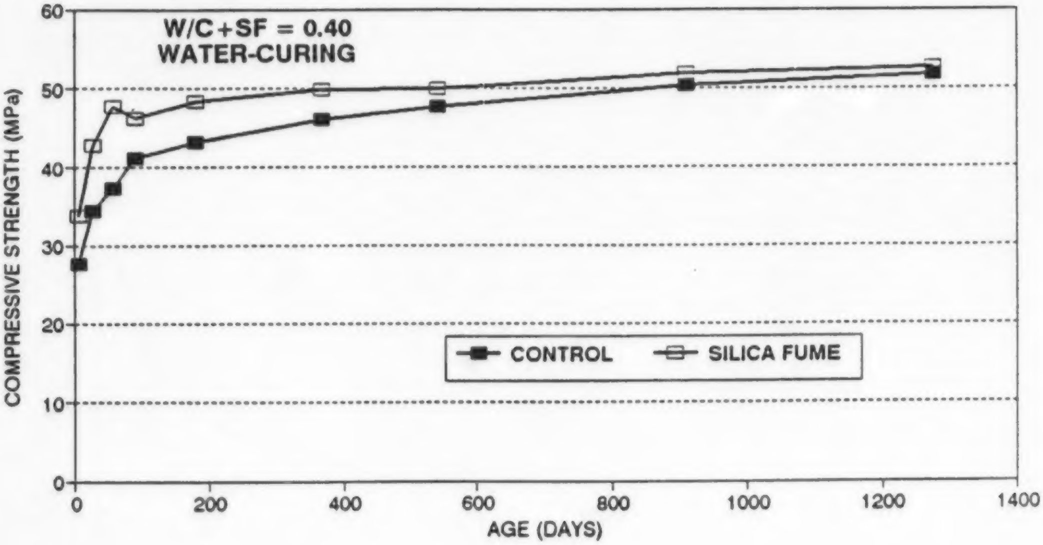


Fig. 12 - Compressive strength development of water-cured concrete - W/C+SF = 0.40. From reference 30.

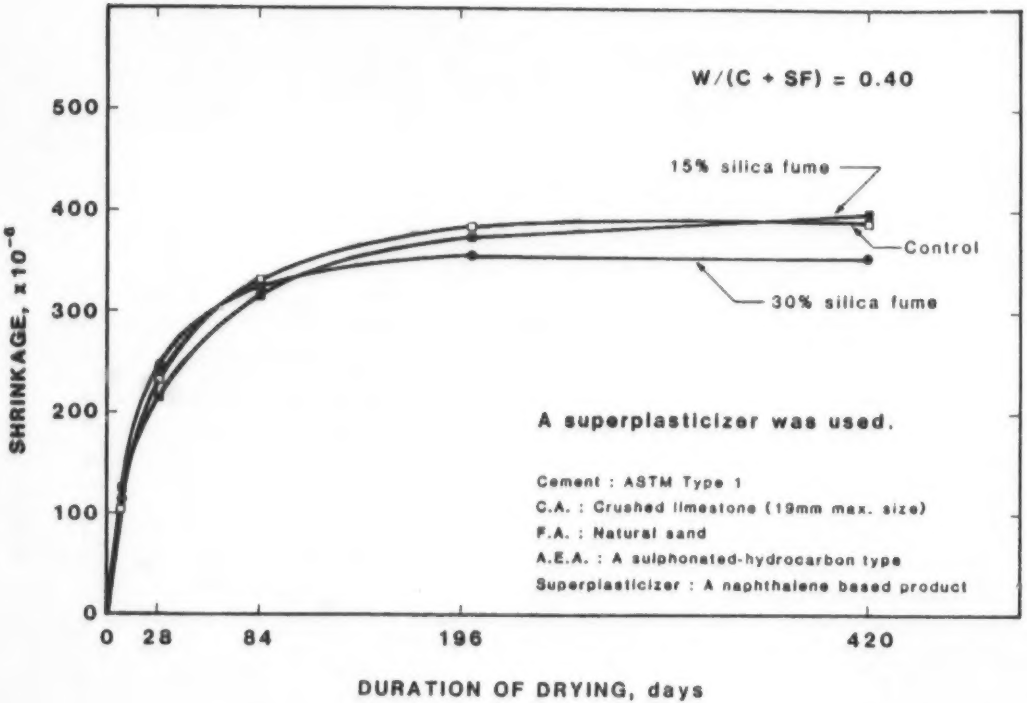


Fig. 13 - Relation between shrinkage and duration of drying for concrete: W/(C+SF) = 0.40. From reference 14.

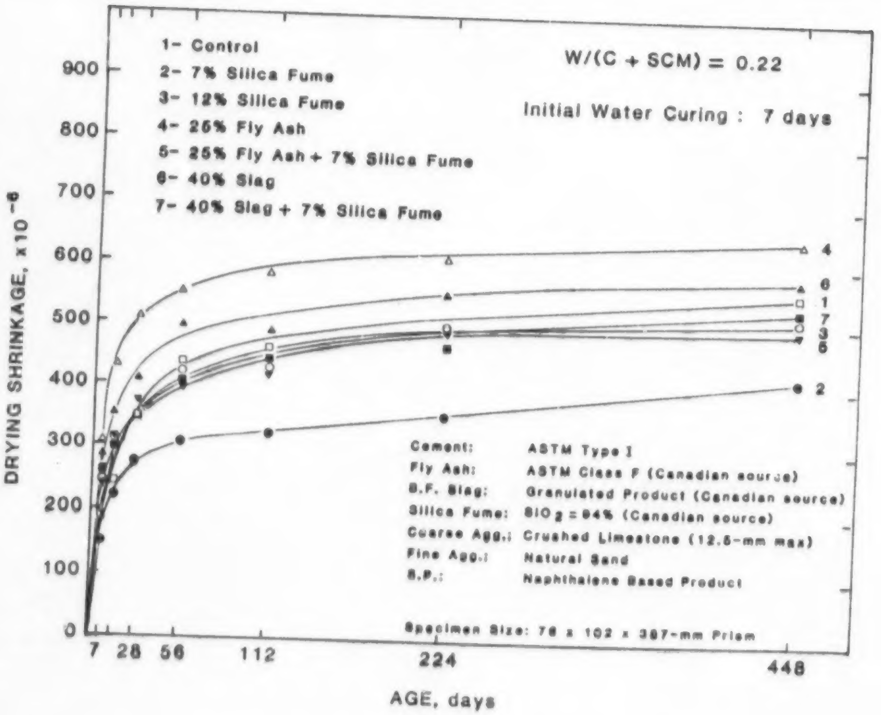


Fig. 14 - Drying shrinkage of concrete prisms. From reference 53.

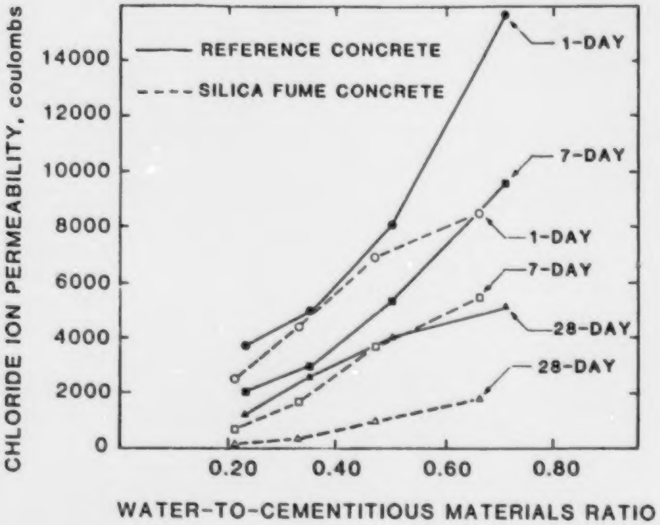


Fig. 15 - Influence of silica fume on chloride ion permeability of concrete. From reference 61.

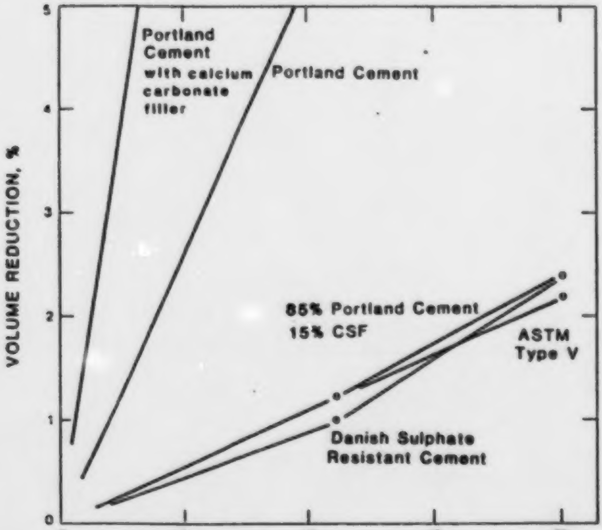


Fig. 16 - Volume reduction of 100 x 100 x 400-mm concrete prisms stored for 20 years in acidic, sulphate-rich water in Oslo alum-shale region. From reference 66.

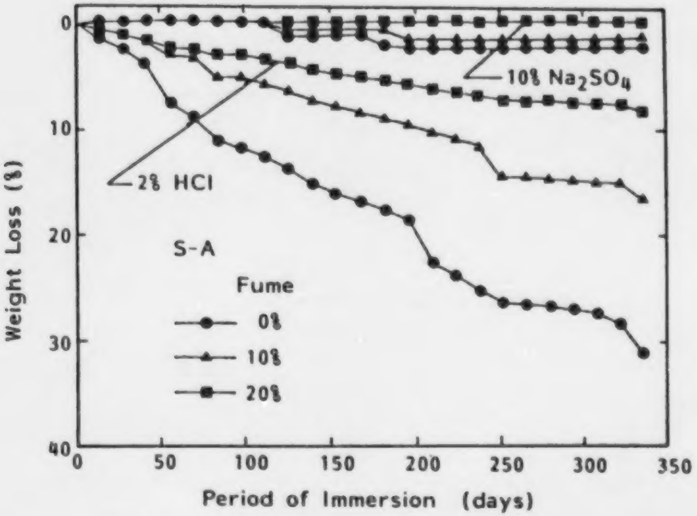


Fig. 17 - Effect of CSF content on weight loss due to 10 percent Na_2SO_4 and 2 percent HCl solutions in concrete. From reference 67.

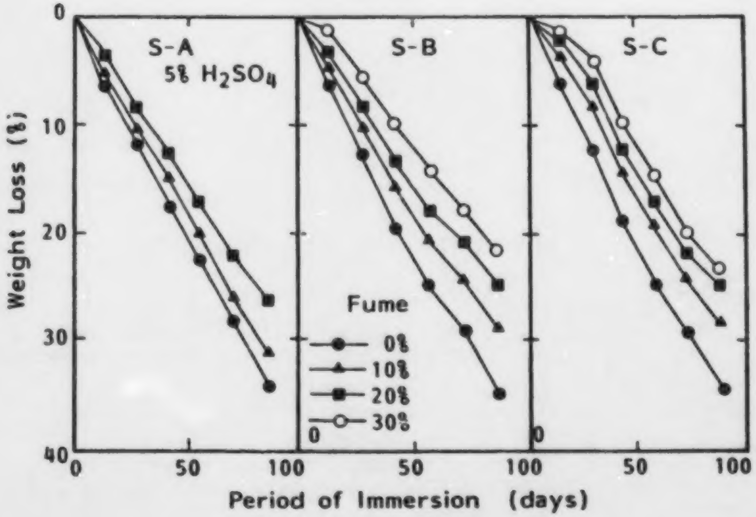


Fig. 18 - Effect of CSF content on weight loss due to 5 percent H_2SO_4 solution in concrete. From reference 67.

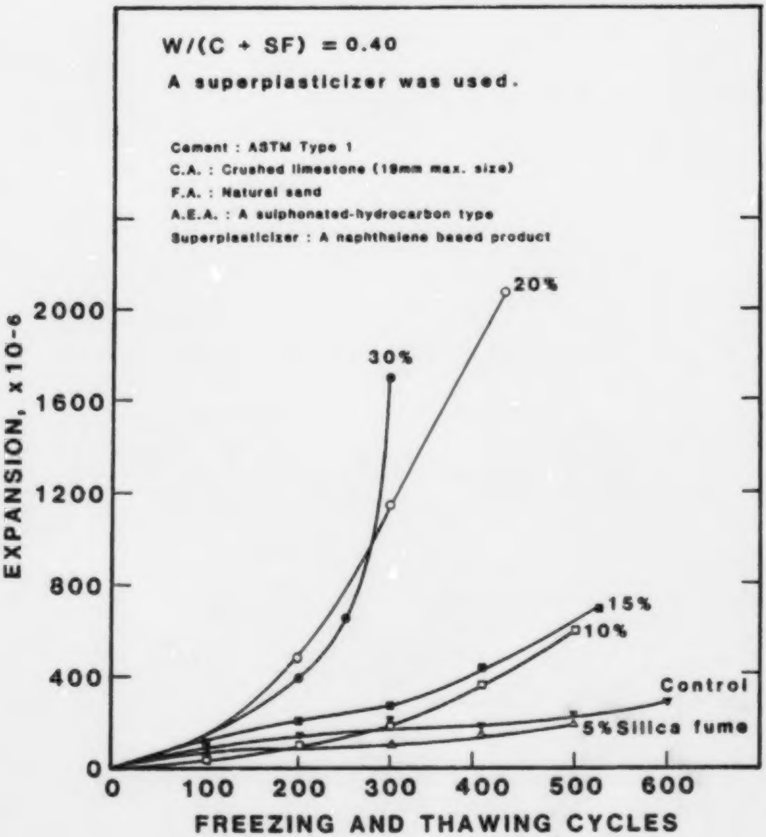


Fig. 19 - Expansion of test prisms after freezing and thawing exposure (ASTM C666 Procedure A). From reference 14.

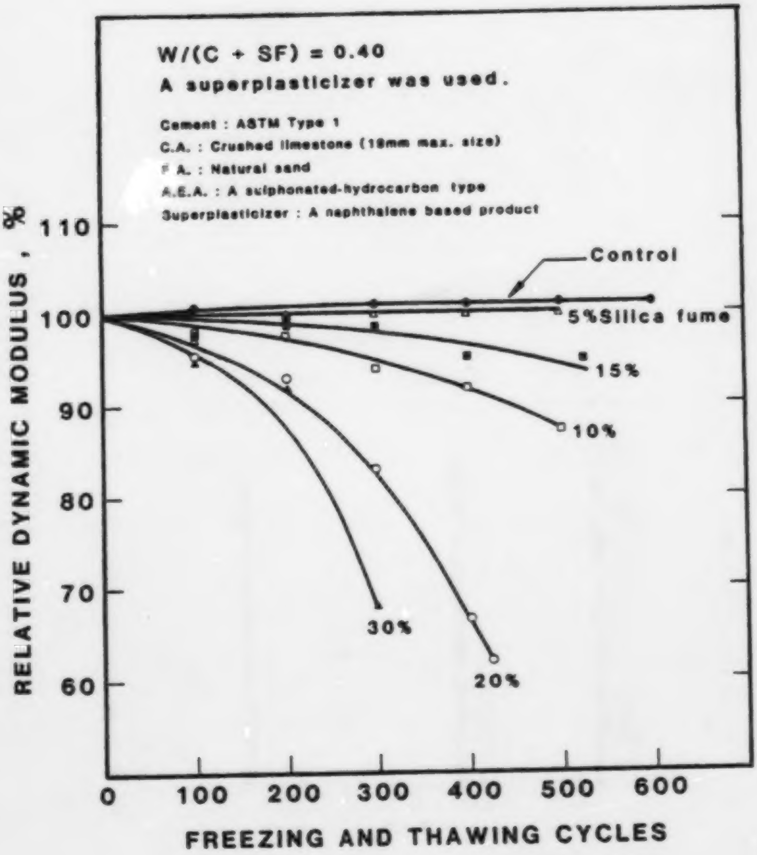
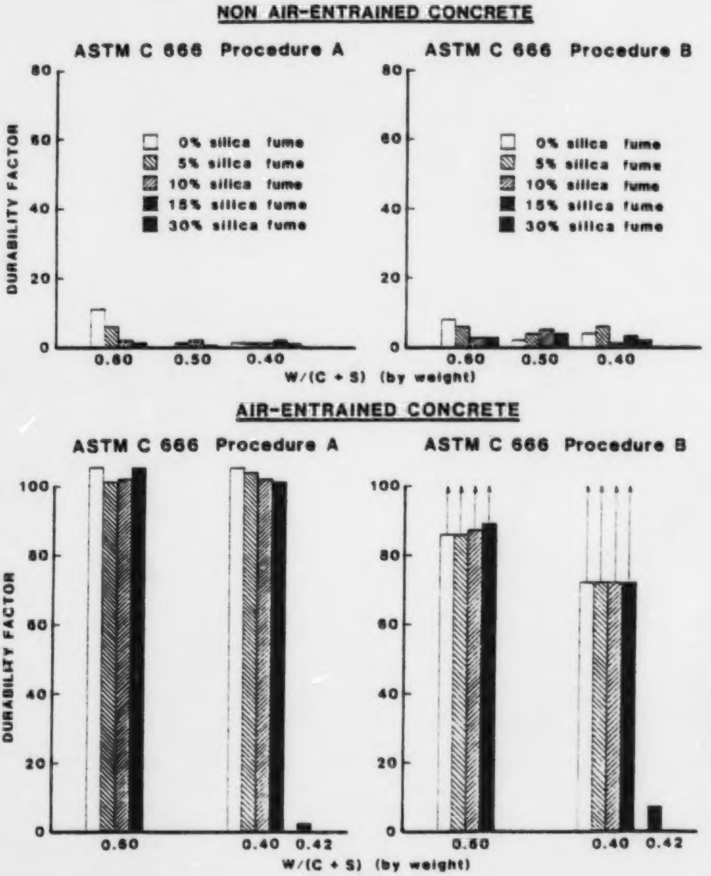


Fig. 20 - Relative dynamic moduli of test prisms after freezing and thawing exposure (ASTM C666 Procedure A). From reference 14.



Cement : ASTM Type 1
C.A. : Crushed limestone (19mm max. size)
F.A. : Natural sand
A.E.A. : A sulphonated-hydrocarbon type
Superplasticizer : A naphthalene based product

Note: Arrows indicate that durability factor would have been higher if test equipment had not broken down.

Fig. 21 - Durability factors for nonair-entrained and air-entrained concrete. From reference 37.

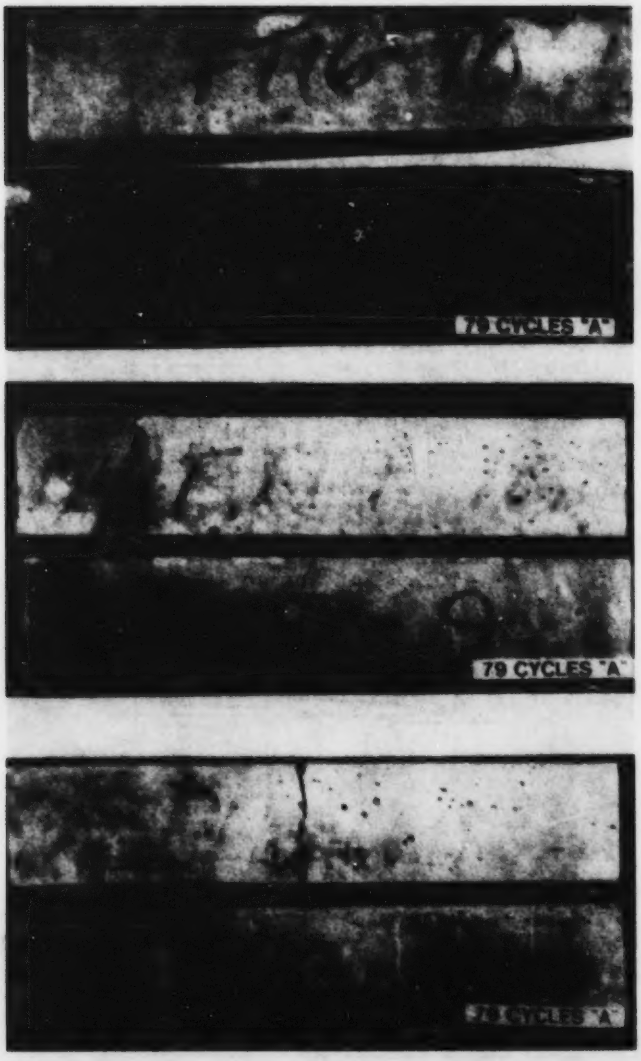

79 CYCLES "A"
79 CYCLES "A"
79 CYCLES "A"

Fig. 22 - Nonair-entrained concrete prisms with and without condensed silica fume after 79 freezing and thawing cycles (ASTM C666 Procedure A). All W/C ratios = 0.25. (Top) 0% silica fume, (middle) 10% silica fume, (bottom) 20% silica fume. From reference 78.

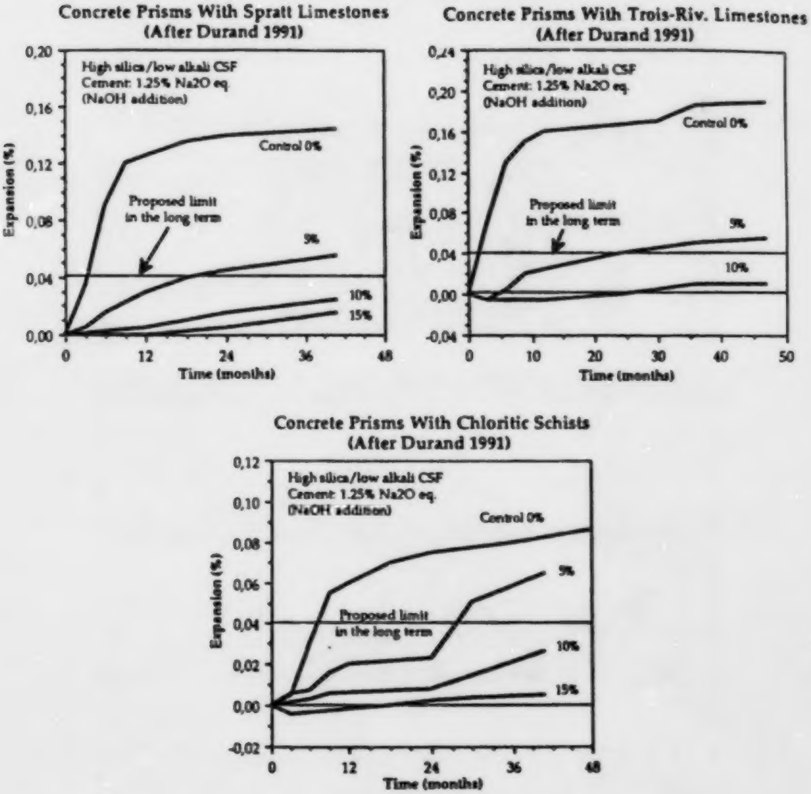


Fig. 23 - Results of expansion tests on concrete prisms (CSA Method) made with three reactive aggregates from Ottawa (Spratt Limestone), Trois-Rivières (limestone) and Sherbrooke (Chloritic schist), respectively, and various amounts of a high silica/low alkali condensed silica fume. For all mixes, the alkali content was increased to 1.25% of the mass of cement (Na_2O eq.) by adding NaOH to the mix water. After 1 year, all samples satisfied the CSA limit of 0.04%. After two years, 5% CSF was no more enough for the two lime-stones, while the prisms made with the chloritic schist and containing 5% and 10% CSF started to expand quite rapidly. From reference 83.

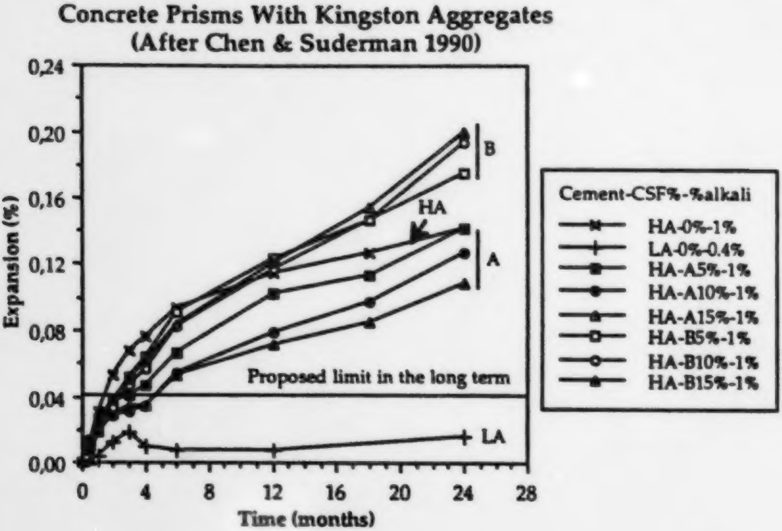


Fig. 24 - Results of expansion tests on concrete prisms (CSA method) containing a reactive argilite from Ontario and various amounts of two condensed silica fumes, one high silica/low alkali (CSF-A) and one low silica/high alkali (CSF-B). The cement used (HA) contained 1.01% alkali (Na_2O eq.), except for the low alkali control (LA: 0.42% Na_2O eq.). The two CSF presented high expansion for any content. Even more, the alkali-rich variety (CSF-B) produced expansion greater than the control (HA), and the higher the CSF content, the higher was the expansion. From reference 84.

CANMET PUBLICATIONS ON CONCRETE

Supplementary Cementing Materials for Concrete

Editor: V.M. Malhotra

This volume brings together seven comprehensive state-of-the-art papers on supplementary cementing materials for use in concrete. Internationally recognized experts from Canada, Norway and the United States were invited to contribute papers on the subject.

The book opens with a paper on natural pozzolans, a material which has been in use for over 2000 years. This is followed by a paper on fly ash, a pozzolanic by-product of the combustion of pulverized coal in thermal power stations, which is finding increasing acceptance for use in concrete the world over. The third paper deals with a worldwide review of condensed silica fume. This material is highly pozzolanic and is a by-product resulting from the reduction of high purity quartz with coal in electric arc furnaces in the production of silicon and silicon alloys. The next four papers deal with ferrous and non-ferrous slags. The ground granulated blast-furnace slags, though in use in Europe for over seventy-five years, are relatively new in North America and offer great potential for use in concrete. The non-ferrous slags, particularly copper, zinc, and lead slags, are of interest to Canada because of the large volumes available in the country.

Fly Ash in Concrete

E.E. Berry and V.M. Malhotra

This volume presents a state-of-the-art review of the principal advances in research, development and practical applications of fly ash in concrete that have been made during the period 1976-1984. A summary of the main technical features of relevant national standards and specifications from a number of countries are described and their shortcomings are discussed. Recommendations are made with regard to aspects of fly ash technology requiring further research.

Progress in Concrete Technology

Editor: V.M. Malhotra

Over the past two decades research in government, industry and university laboratories has resulted in major developments in concrete technology. New types of lightweight aggregates, polymer-, sulphur-, and sulphur-infiltrated concretes; fibre-reinforced concrete; and high-range water reducers/super plasticizers are examples. Equally impressive developments have also taken place in the evaluation of fresh and in-place concrete.

The purpose in publishing "Progress in Concrete Technology" has been to bring together in one volume in-depth reviews on the above developments. This was achieved by inviting internationally recognized authorities to contribute comprehensive state-of-the-art papers on selected subjects.

Nondestructive Methods for Testing Concrete

V.M. Malhotra

This monograph deals with nondestructive methods for testing concrete. Different methods of hardness testing are briefly mentioned and the Schmidt test hammer, based on the rebound principle, is described in detail. Dynamic tests utilizing resonant frequency, mechanical sonic and ultrasonic pulse velocity are described and their various applications and limitations are discussed. A radioactive test method employing X-ray penetration tests is briefly mentioned. The principle of gamma radiography is given and some of its application are described.